MASTER OF SCIENCE

Circular dry cement inclusions
a novel approach to stabilising clay soil

Stuart, Kristofer Martin

Award date:
2011

Circular dry cement inclusions

a novel approach to stabilising clay soil

Kristofer Martin Stuart

2011

University of Dundee

Circular Dry Cement Inclusions:

A Novel Approach to Stabilising Clay Soil

By Kristofer Martin Stuart

DECLARATION

I hereby declare that the following Thesis has been composed by me; that, unless stated, all references cited have been consulted, that all work of which the Thesis is a record has been carried out by myself, and that it has not been previously been presented or accepted for a higher degree.

Kristofer. M. Stuart

Kristofer Martin Stuart

CERTIFICATE

This is to certify that Kristofer Martin Stuart has completed his research under our supervision and that he has fulfilled the conditions of the relevant Ordinance and Regulations of the University of Dundee, so that he is qualified to submit this Thesis in application for the Degree of Master of Science.

Dr M. R. Jones

Reader, Dean of School

Concrete Technology Unit

Division of Civil Engineering

University of Dundee

Dr M. J. Brown

Lecturer

Geotechnical Engineering Research

Division of Civil Engineering

University of Dundee

DEDICATION

I would like to take this opportunity to personally thank and dedicate this report to my girlfriend Susan, who has always been there to support and motivate me throughout my time at university. There are no words to describe how much she has helped me through difficult times and has always been there to keep things in perspective and enable me to see the bigger picture. I love her with all my heart and I am blessed to have her in my life.

I would also like to dedicate this project of work to my family, especially my mum and grandma, who have been extremely supportive throughout the entire duration of this work. Their advice, patience and constant assurances have kept me sane. My achievements are compliments of their hard work and advice throughout my life and I am tremendously lucky to have such a fantastic family. Without them I never would have had the belief to complete university and reach my full potential. Thank you.

I would also like to thank my uncle Mark; who was the one who inspired me to become an engineer. His guidance and the continuous help that he has provided to me over the years, has been fantastic. I hope that one day I can repay his faith and become the best I can be.

ACKNOWLEDGEMENTS

I would like to take this opportunity to personally thank all those who have participated to this research project. First and foremost, I would like to thank my supervisors; Dr Rod Jones and Dr Michael Brown for all their invaluable expertise, input, guidance and continuous support throughout the years. Without their help and efforts this project would not have been possible and I am grateful that I have had the opportunity to work with them during this research – even if I did dread some Thursday morning meetings. I hope this research goes some way towards repaying them for the faith that they have shown in me throughout my time at the university.

I would also likely to thank the technical staff at the University of Dundee, most noticeably Mark Truswell, who has been fantastic with his advice and efforts during the experimental stages of this project. A special mention must also be given to Dr Laszlo J. Csetenyi, Dr Li Zhing, Dr Tom Dyer, Dr Michael McCarthy and Dr Moray Newlands for all their efforts in the laboratory and advice throughout.

ABSTRACT

Limited adoption of research and development has led to Britain lagging behind many European and far-east countries in terms of geotechnical on-site innovations. This, coupled with the fact that construction costs are considered to be 30% higher than they should be for what is delivered (Egan, 1998), has led to a general change in the countries attitude and approach towards future construction. This project undertook an initiative approach to soil stabilisation, with a key aim of establishing whether or not a deliberate inclusion of dry Portland cement would induce consolidation in a clay soil, by transferring and fixating pore water into the cement and changing the structure of the surrounding soil.

Laboratory prepared kaolin clay was the soil material adopted for testing purposes, with soil moisture contents of 40% and 60% and a range of curing times up to and including 28 days under investigation. Two cement materials: i) Portland cement (BS EN 197: CEM 1) and ii) Calcium Sulfoaluminate Cement (CSA) were investigated in order to determine the influence of cement material.

Moisture content samples and hand shear vane tests were undertaken to determine the strength and moisture changes in the surrounding soil as a result of the cement inclusion utilising the soils pore water. The penetration of water through the cement inclusion was monitored by the cements degree of hydration using Thermogravimetric testing (TGA). The porosity of the cement was determined using Mercury Intrusion Porosimetry (MIP) and Nitrogen Adsorption. To further investigate the ability of cement inclusions to absorb pore water, capillary sorption tests were performed over a range of curing times up to 28 days. All tests were performed to laboratory scale, with an empirical approach to the consolidation effect being undertaken.

Very early on in the work it was found that cement, contrary to the literature and the general debate around the approach, did absorb pore water and hydrate in such a manner as to enhance the characteristics of the surrounding clay soil. Increased curing time was seen to benefit the system; with the clay shown to continually absorb water up to 28 days. All cement within the inclusions was seen to have access to water and fully participated in the improvement process.

Work was then progressed onto investigating the effects of the inclusion diameter, which suggests that larger inclusions require longer curing times to hydrate due to sorption and diffusion leading to an 'onion skin' effect. An investigation into grouped inclusions organised in a number of arrangements was also performed, focusing on the spacing between inclusions and the area replacement ratio.

It is speculated that a simple 1-dimensional rig could be designed in order to investigate the dewatering effects experienced in both normal and overconsolidated clay soil, in relation to a layer of dry cement.

Work concludes with some practical suggestions for future research; intended to further understand the advantages and limitations to this proposed system.

TABLE OF CONTENTS

TABLE OF FIGURES

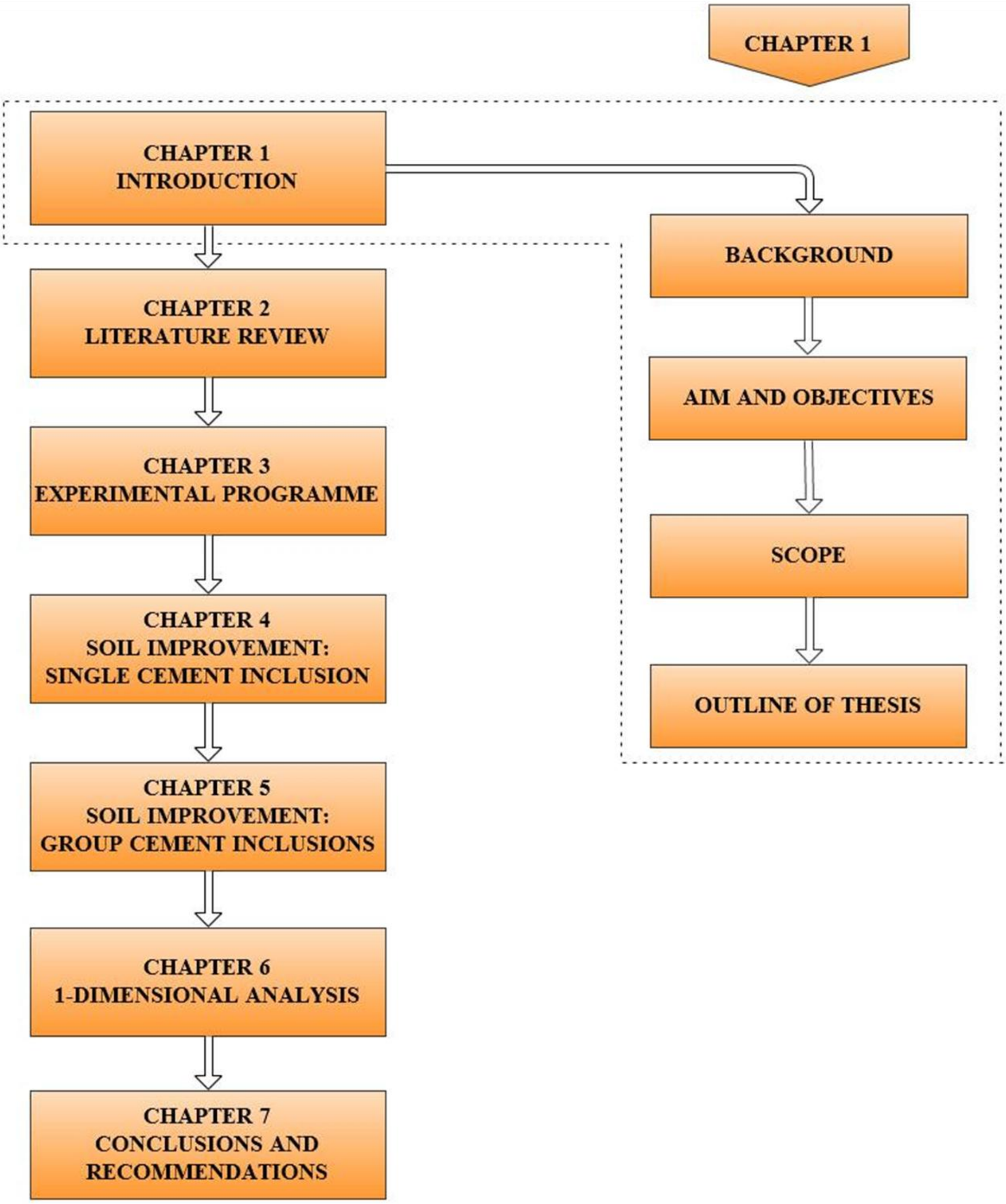
LIST OF TABLES

LIST OF ABBREVIATIONS

NCL	–	Normal Consolidation Line
OC	–	Overly consolidated soil
OCR	–	Overconsolidation Ratio
LI	–	Liquidity Index
CMC	–	Controlled Modulus Columns
DJM	–	Dry Jet Mix
CSA	–	Calcium Sulfoaluminate cement
PC	–	Portland cement
C-S-H	–	Calcium Silicate Hydrate
w/c	–	Water/cement ratio
MIP	–	Mercury Intrusion Porosimetry
TGA	–	Thermogravimetric Analysis

LIST OF SYMBOLS

w	-	Moisture content (%)
σ_v	-	Total stress (kN/m^2)
σ'_v	-	Effective stress (kN/m^2)
u	-	Pore pressure (kN/m^2)
e	-	Voids ratio
e_o	-	Initial voids ratio (Assumed when $\sigma'_v = 1$)
e_k	-	Final voids ratio
C_c	-	Gradient of Normal Consolidation Line (NCL)
C_s	-	Gradient of overconsolidated (swelling) line
C_u	-	Undrained shear strength (kN/m^2)
C_3S	-	Tricalcium silicate
C_2S	-	Dicalcium silicate
C_3A	-	Tricalcium aluminate
k'	-	Intrinsic permeability
η	-	Dynamic viscosity of fluid ($N\ s/m^2$)
Q	-	Flow rate (m^3/s)
A	-	Cross sectional area (m^2)
v	-	Apparent velocity of flow
ρ	-	Density of fluid (kg/m^3)
g	-	Acceleration due to gravity (m/s^2)
i	-	Increase in mass per unit cross sectional area in contact with water divided by density of water (mm)
t	-	Time (minutes)
s	-	Sorptivity ($mm/mm^{0.5}$); where: $mm/mm^{0.5} = 1.29x10^{-4}\ m/s^{0.5}$
D	-	Effective diffusion coefficient (m^2/s)
L	-	Thickness of sample (m)
$\frac{dc}{dL}$	-	Concentration gradient (kg/m^4)
J	-	Mass transport rate (kg/m^2s)
G_s	-	Specific gravity
r	-	Radial distance (mm)



"If we knew what it was we were doing it would not be called research would it?"

Albert Einstein (1879-1955)

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Construction costs have been under scrutiny since the Egan report (1998) as they have been considered to be 30% higher than they should be for what is delivered. There are numerous reasons for this; however an ongoing problem is the limited adoption of a research and development approach. Of relevance here is geotechnical practice, which has perhaps lagged behind the rest of Europe and the far-east, particularly with respect to on-site innovation.

One recent development now adopted in the UK is Controlled Modulus Columns (CMC), originally developed in France by Menard. CMCs can best be described as discrete, vertically installed semi – rigid mortar inclusions, which reach a predetermined stiffness ratio between the column and surrounding soil; creating a complex soil + inclusion composite material (DGI-Menard Inc, 2009). CMCs produce rapid large area ground improvement using a 3 part hollow-stem displacement auger; capable of laterally displacing the soil (Figure 1.1). The column is formed using wet concrete of known water/cement ratio, which allows the modulus of the soil-cement column to be constant along its length.

Private communications (Jones, 2010) detailed the use of CMCs at Dartford Park, which was the first project in the UK to utilise CMC technology. At this same site the University of Dundee was assisting and advising the contractor on the use of foam concrete. Having looked at the operation of CMCs with ready mixed concrete, a project idea was formulated that it may be advantageous to use dry cement rather than the injection of slurry. The basis for this was the theoretical ability for the cement to consolidate the surrounding soil by 'removing' pore water in order to hydrate. In addition, the hydrated cement column would act in a similar manner to a CMC array.

a) CMC augers used to stabilise soil at Dartford Park, UK

b) Overview of equipment necessary for installation of CMCs at Dartford

c) CMC 3 Part Auger

d) Installation of CMC column

e) Extraction of CMC auger (shows minimal soil being brought to surface)

f) Completed CMC Column

Figure 1.1: CMC Installation at Dartford Park (Jones, 2010)

In addition, advantages of using dry cement injection is that work can be started and stopped at any point without having to either clean the equipment or waste any unused concrete.

This project was, therefore, devised to study the potential for dry injection of cement in a CMC-type array to consolidate *in-situ* soil, as well as provide the effect of introducing stiff inclusions. Clearly there was a degree of risk in this novel approach as there is almost no parallel in current geotechnical engineering. The nearest precedent is deep soil mixing with lime or cement but this is a very different process. As there is no mechanical mixing with this proposed ground improvement technique there was concern that the amount of pore water that could be 'removed' by cement hydration would be limited, particularly if the 'skin' of the inclusion effectively sealed the soil from the unhydrated core. This project addresses this hypothesis.

If this method of ground improvement is proved to be successful in delivering its aim, the possible applications for its use are best considered in remote, developing areas where disasters involving soil saturation have arisen. Pakistan is an example where significant geotechnical costs have prevented proper stabilisation to slopes and levees taking place; resulting in catastrophic floods, as too are the recent disasters in Hungary and New Orleans. The possibility of dry inclusions being installed at a relatively low cost and in a far quicker time in comparison to more established ground improvement techniques would be beneficial to these situations; providing the aims of this study are met.

1.2 AIM AND OBJECTIVES

Given the background information above, the aim of this current research was to test, using an experimental approach, whether the hypothesis of simple placement of a dry cement inclusion in clay soil could produce an enhancement of the soil characteristics, in particular strength, via consolidation. If so what are the boundaries to which this can be realised. The following key objectives are defined to achieve the aim of this study:

i. Review established stabilisation techniques that involve the use of dry cement to utilise pore water from the soil and explore the theory behind their ability to improve the soil.
ii. Devise scale experimental test methodology and carry out laboratory tests to determine if soil improvement can be achieved using a dry cement inclusion.
iii. Determine whether cement hydrates to the inclusion core as a result of its interaction with pore water in the soil.
iv. Examine the radial influence of a dry cement inclusion and its ability to enhance the soils strength.
v. Examine the influence of the inclusion diameter on radial improvement in the soil.
vi. Design a one-dimensional rig capable of providing both physical and visual inspections to be carried out with respect to clay soils interaction with a dry cement inclusion, and perform tests in the rig focusing on a range of soil loading conditions.
vii. Develop an empirical description to explain the general effect of soil dehydration through cement hydration.
viii. Assess the performance and environmental benefits of incorporating a different material in place of Portland cement to form the inclusion.

1.3 SCOPE

Considering the substantial amount of cements and by products available, and the endless material portions which could be investigated, it was decided to use 100% Portland cement to form the inclusions. This decision was based on Portland cement being the most widely used material in modern construction, coupled with its use in a number of established techniques to stabilise soft clay soils e.g. lime-cement columns and Japanese dry deep mixing method. An investigation involving a single CSA cement inclusion was undertaken in order for a material comparison to be presented in the study; however no other material types were considered.

Pure kaolin clay was used to represent the clay soil for the purpose of this study, due to its low cost and high plasticity. It was also selected as it was locally sourced therefore easily accessible throughout the entirety of the investigation. The kaolin was supplied from an industrial source in powder form, therefore required the addition of water and a degree of mixing to obtain a homogenous consistency representative of *in situ* clay soil. The test moisture contents used for the soil were established by initial testing to ensure the powder could be mixed to a homogenous consistency. In this case 40% and 60% were tested, which were arbitrary values.

An auger design and recommendation for injection pressures for the dry cement were intentionally left out of the scope of this project. This was simply because the hypothesis of whether dry cement would work in place of wet slurry had to be confirmed to work. Full scale trials were also not considered due to the complexity of auger design.

The durability of the cement inclusions was not investigated; therefore no details regarding performance against sulfate attack are presented. Also the strength of the inclusion was not investigated as it is established with CMC technology, instead the soil mechanics of the soil dewatering effect, as a result of the dry inclusion utilising pore water from the soil, was considered.

1.4 OUTLINE OF THESIS

As mentioned, this Thesis aims to investigate whether the hypothesis of simple placement of a dry cement inclusion in clay soil could produce an enhancement of the soil characteristics. This project work will use an experimental approach and will focus on increasing soil strength by a process of dewatering; caused by dry cements ability to utilise water from a soils matrix.

Chapter 2 critically reviews the literature available on established ground improvement techniques – lime-cement columns, stone column, CMC technology; and the theory behind their ability to stabilise clay soils. In addition, the fundamental process of cement hydration is presented to aid the understanding of cements ability to utilise water.

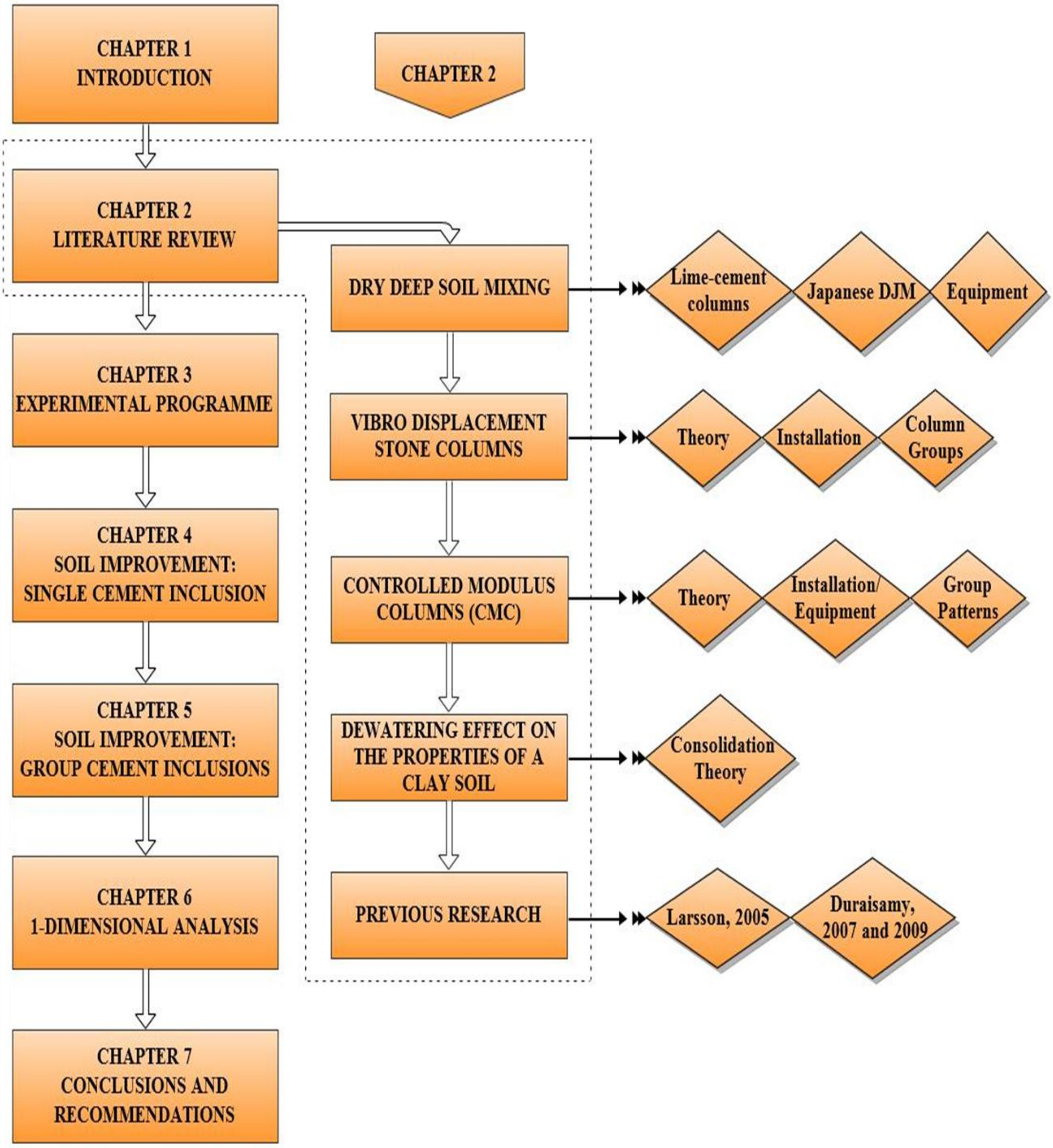
From the information gained in the literature, Chapter 3 presents the relevant materials, testing procedures and background theories developed by the Author in analysing the performance of dry cement inclusions. In addition to this a detailed account of the experimental methodology is available.

The results obtained from investigating single cylindrical inclusions, of varying diameter and cured for different periods, are presented in Chapter 4. An investigation into the inclusions performance in two different soil conditions is also available, with detailed visual observations presented on behalf of the Author.

Chapter 5 investigates the performance of the 'dry cement inclusions' in three group arrangements - selected from information provided in the literature in Chapter 2.

A one-dimensional analysis is performed in Chapter 6 with both physical testing and visual inspections performed using specially designed rig equipment. This Chapter will focus its attention to consolidated soils; subjected to different loading conditions, as well as investigating the cements ability to utilise moisture from the clay soil when there is a reduction in overburden pressure and an absence of water.

While each Chapter includes a summary, Chapter 7 will provide an overall conclusion of the 'dry cement inclusions' ability to stabilise clay soil. Detailed findings and practical implications of the work allows conclusions to be reached based on the results of this research. However, recommendations are provided by the Author for areas where further research should be carried out to further understand the application of dry cement inclusions.



"Research is what I'm doing when I don't know what I'm doing"

Wernher von Braun (1912-1977)

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

As noted in Chapter 1 the nearest applications to the system being studied in this research work are lime-cement columns, stone columns and CMCs. This Chapter critically reviews the published literature available in regard to these established techniques; focusing on the theory behind why the soil improves, the equipment used for installation and their apparent advantages/disadvantages. Similar applications and research involving the investigating of dry binder as a means of dewatering soil are also under review; considering the hypothesis provided in Chapter 1. A brief description of cement hydration and the stabilisation of clay soil as a result of dewatering is also presented.

2.2 DRY DEEP IN-SITU SOIL MIXING METHODS

Dry deep *in-situ* soil mixing is a rapidly developing and widely accepted ground improvement concept used to stabilise and provide column-type reinforcement to soft soils (Stavridakis, 2006). It is classified as a ground improvement system, rather than a traditional piling system, as applied loads are carried by the mechanical interaction of the columns with the surrounding soil (Arulrajah, 2009); not solely by the column itself. It has been used extensively in Japan and the Nordic countries for road and railway embankment applications since its genesis in 1967 (Hryciw, 2007), however its application has increased significantly in the US, Germany and the UK in recent years (Topolnicki, 2004).

The term 'soil mixing' refers to techniques which incorporate a process of mechanically mixing *in-situ* soil with a dry stabilising binder; in order to utilise moisture from the native soil and initiate a hydration reaction. The reason for performing the mixing process is to avoid large aggregate formation, as this restricts the ability of the binder to react with the soils moisture resulting in a poor quality mix. The mixing tools generate high compressive and shear forces in the soil which help to break up the aggregates and contribute to the release of water from the soil and provide the best possible conditions for a chemical reaction to take place (Larsson, 2005). The result is a hardened stabilised mass of increased shear strength, reduced compressibility and reduced permeability.

There are two commonly used techniques for performing dry deep mixing. These are the lime-cement column and Japanese Dry Jet Mixing (DJM) methods. A brief discussion focusing on both the theory behind the techniques ability to stabilise the soil and the installation practices adopted on site, as well as the advantages and limitations associated with each of the two techniques, is provided in the following literature.

2.2.1 Lime-Cement Columns

Lime-cement columns are predominantly used in the Nordic countries to stabilise soft clay soils and can accredit their improvement to the surrounding soil strength through hydration, ion exchange, flocculation and pozzolanic reaction (Bergado, 1996). They are formed by mixing a combination of lime and cement with the *in situ* soil; in order to achieve a uniform soil mass which consisting of no lumps of untreated binder, and rely on small, lightweight equipment providing design strengths in the region of 0.2MPa (Holm, 1999).

The mixing process performed in lime-cement columns is very complex and can influence both the quality of the column and the resultant improvement in the soils strength. However, as well as mixing, the strength improvement is highly dependent on factors such as characteristics of the binder, the characteristics of the soil and the curing conditions. Huat (2009) provides a detailed account of each of these influential factors and should be referred to for more information:

Table 2.1: Summary of factors to be considered when installing lime-cement columns (Huat, 2009)

Installation Process	Mixing Process
Geometry of the mixing tool	Rheological properties of the unstabilised soil and the mixture
Retrieval rate	
Rotational speed	Type and amount of binder
Feed pressure and the amount of air	*In situ* stress condition during the curing period
The machine type and driver	
In situ stress at time of installation	Trials involving different test methods

The major advantage of using lime-cement columns to stabilise soft soil is that the shear strength can be improved by a magnitude of three to four times in comparison to the untreated soil; with the largest shear strength improvement expected to occur between moisture contents of 40-80% (West, 1997). The method is also very flexible since column spacing, column depth and column diameter can be adjusted to the specific problem.

Binders can be applied in either slurry or dry form depending on the nature of the virgin soil, with wet mixing generally preferred as it is more difficult to mix soft soil with dry cement; especially when the cement content is high. However, the ability of a dry mix to utilise moisture from the soil in order to hydrate can lead to an undrained compressive strength improvement 1.7 to 3.2 times greater than by performing wet mix practices (Topolnicki, 2004). In general dry mixing is preferred in site conditions where the water table is high and close to the ground, as this allows the binder to react with soil moisture along the full length of the column (Liu, 2008).

Lime-cement columns were an adaptation to the early lime columns used to stabilise soils in Nordic regions, as incorporating cement into the column mix allowed early strength development problems associated with lime columns to be overcome (Figure 2.1).

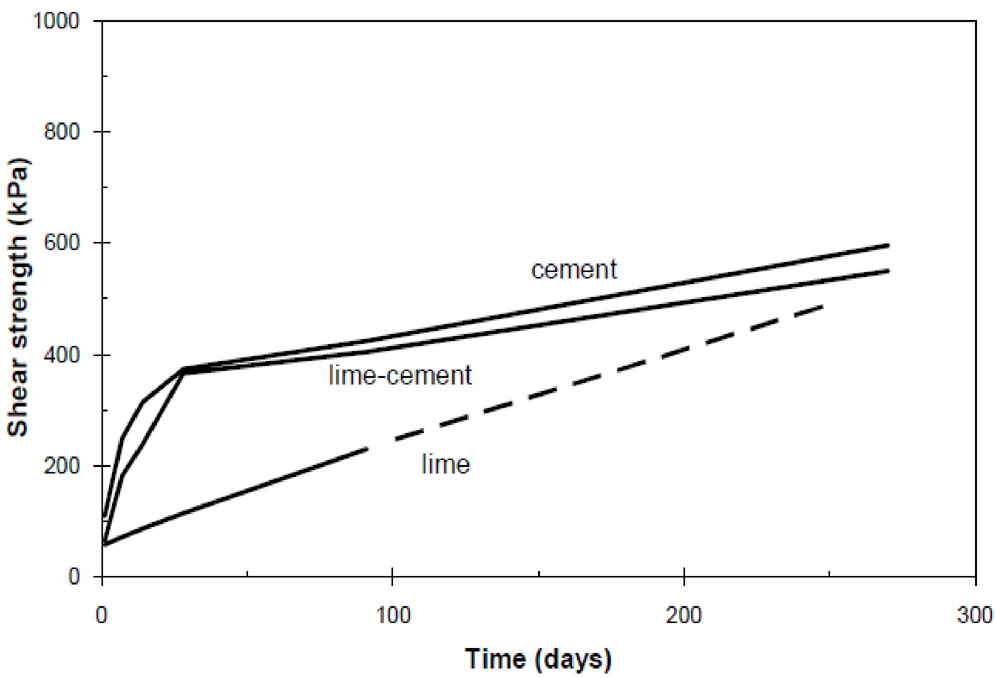


Figure 2.1: Comparison of strength development with time after mixing clay with lime, lime-cement and cement (Åhnberg, 1995)

However, a disadvantage to incorporating cement into the column is that a great deal more mixing and energy is required to achieve a high shear strength that is uniform along the column length (Larsson, 2005); which also has an effect on productivity as it increases the installation time necessary to form the columns. Another drawback to increasing the cement content is that the permeability of the column reduces with increased curing time, which eliminates the ability of the hardened column to act in a similar manner to a vertical drain; as is the case with lime columns.

Investigations into better understanding the factors (Table 2.1) influencing the strength improvement of soils are ongoing, with Larsson (2005) reporting that natural soils when stabilised by lime-cement mixing, are not homogenous along the length of column. This results in different values of shear strength, stress and stiffness being recorded along the column, with an uneven distribution of binder. These all have a direct influence on the behaviour of the column and are not advantageous; as under loading conditions the column fails to act in an efficient manner with different sections acting under different loads.

Construction limitations on the Nordic methods have also been identified with the current mixing equipment unable to function in soils with shear strength greater than 25kPa (in some cases a maximum of 50kPa) i.e. can only be used in very soft to soft soils (Topolnicki, 2004). Another disadvantage is that soil disturbance only occurs directly around the mixing tool with relatively large forces required to overcome the clays resistance; making the energy expended to produce lime-cement columns being considerably higher in comparison to other forms of ground improvement.

Continual developments in the design of lime-cement mixing tools

Although the mixing tools (Figure 2.2) used to form lime-cement columns have been constantly reviewed, tested and improved; in comparison to the initial tools utilised in 1967, there is still a relatively large degree of uncertainty regarding the geometry and efficiency of the existing mixing tools used in Nordic countries today.

The debate centres on the penetration phase of the mixing tool and the subsequent disaggregation of soil particles. At this time the geometry of the mixing tools and paddles facilitate insertion of the tool, so little energy is required for penetration (Larsson, 2005). As a result, little disturbance is experienced by the soil; which is required to produce the shear forces necessary to break up the soil aggregates and allow the binder to fully participate in the hydration reaction. If this does not take place during the penetration stage, then it must ensue during dispersion of the binder, which is current practice. The risk is that too much energy is expended on remoulding the clay rather than obtaining a sound homogenous mix. However, there is no evidence to suggest that complete disaggregation during penetration will lead to a better quality mix with improved strength. Further research to understand and improve the efficiency of the mixing tools and mixing process is being perused in order to clarify these concerns.

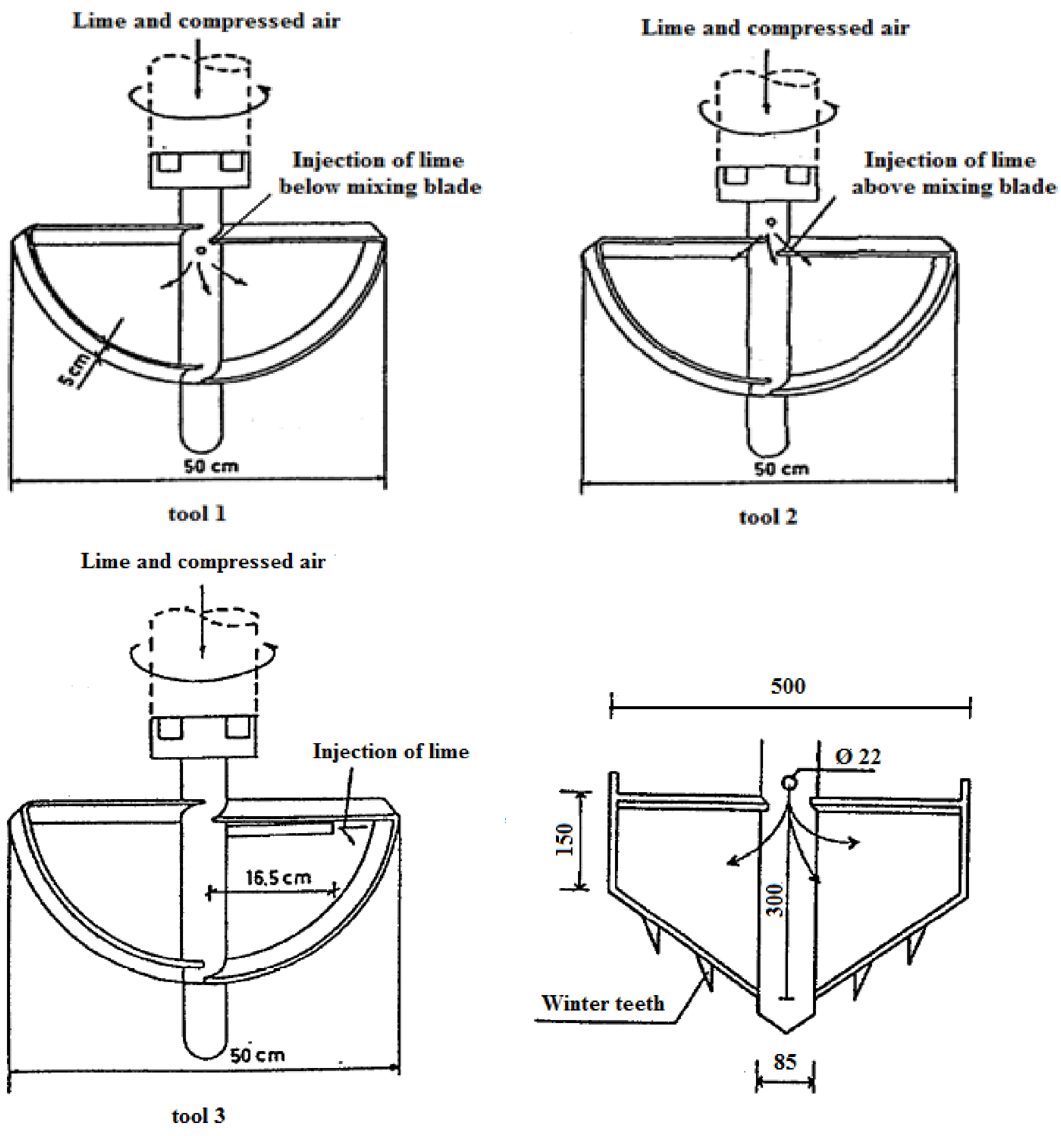


Figure 2.2: Standard mixing tools used to produce lime-cement columns (Stabilator Technical Information, 1992)

2.2.2 'Japanese' DJM

The purpose of the 'Japanese' DJM is similar to the Nordic method with dry binder being mixed with the *in situ* soil to create a hardened mass with improved engineering properties. Again moisture from the soil is utilised in order to produce a pozzolanic reaction which improves the strength with increased curing time. The only differences between this method of stabilisation and lime-cement columns is the equipment and cutting tools employed during installation. Using large scale equipment the Japanese DJM can achieve a uniform mass; consisting of no unreacted binder and constant moisture content along the column length, capable of providing treated soil with unconfined compressive strengths in excess of 0.5MPa (Holm, 1999).

The technique is classified as a 'pneumatic binder feeder system' which penetrates the soil via a hollow, rotated mixing shaft tipped with some type of cutting tool, to depths of 16 to 33 metres. Typically two mixing tools of diameter 1.0 to 1.3 metres are utilised with either one or two mixing shafts (Figure 2.3), which provides the added benefit of quicker construction time; estimated that 30 to 40 columns can be produced in an 8 hour shift (Topolnicki, 2004).

The method involves pneumatically delivering a dry powdered binder, by compressed air, from a pressurised tank through a series of connecting pipes, down the central hollow shaft, to the mixing machine (Figure 2.4). The dry binder typically consists of 30% to 40% cement content and can be injected during the following stages; penetration, extraction or both (Holm, 1999). Injecting the binder during extraction of the mixing shaft is suggested by (Topolnicki, 2004), as the upper mixing blades direction of rotation is reversed causing a cavity space to form. The binder is then horizontally injected into the cavity and subsequently mixed with the soil via the lower mixing blade. Faster rotation speeds increase the volume of the cavity and creates a vacuum which facilitates the process when manufacturing large-diameter columns (Chida, 1982).

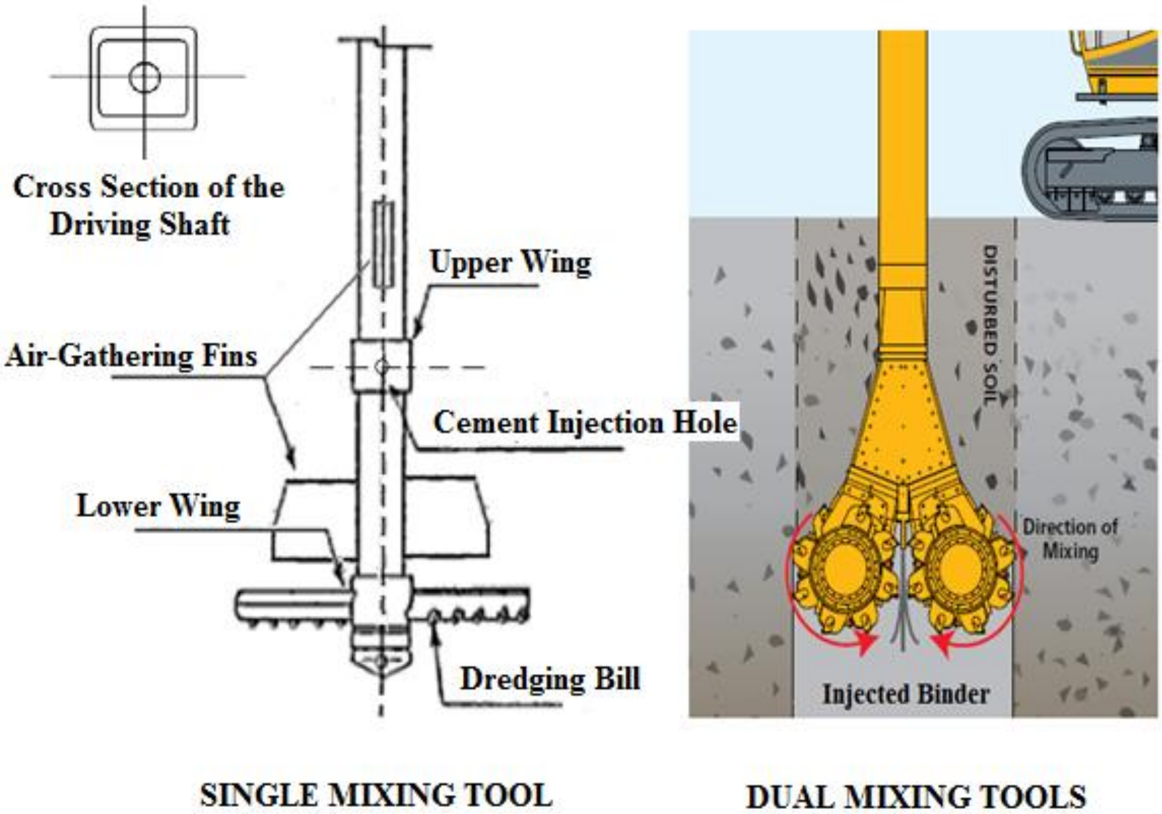


Figure 2.3: Single and dual mixing tool utilised in deep mixing (Holm, 1999)

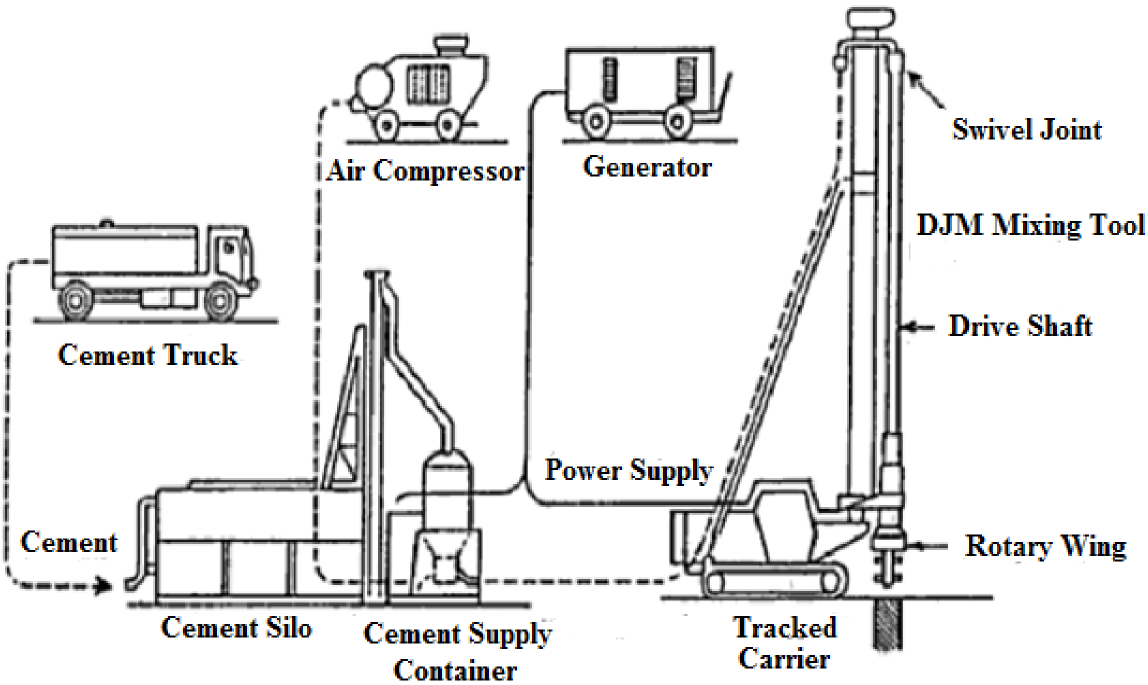


Figure 2.4: Equipment and process for installing DJM mixed columns (Topolnicki, 2004)

Limitations in using this method of dry mixing arise in soils where the moisture content is near or above their respective plastic limit (relatively dry and hard soil). In these conditions, there is not enough water present for a full reaction to take place between the cement and soil, which causes the strength development of the soil-cement column to be unsatisfactory. This problem has somewhat been overcome by the addition of water during insertion of the mixing tool, which allows the soil profile to obtain a consistent water content (LCTechnology, 2002).

The technique is permitted in soft to firm soils where the maximum soil shear strength does not exceed 70kPa (Topolnicki, 2004). Similarly to the Nordic method, this strength limitation is due to difficulties in distributing the cement uniformly throughout the mass, as current machines are unable to adequately divide and mix the heavy clay layers to achieve a homogeneous mix (Ingles, 1987). Difficulties in mixing also occur with increasing cement content; which can lead to much greater variation in shear strength and stiffness along the length of the column.

As identified above the Japanese DJM is not a viable solution in certain ground conditions, however it can be an attractive technique over other forms of ground modification where the above limitations are satisfied. The following is a summary of the project conditions where deep mixing is favourable over other techniques:

- Ground is neither stiff nor very dense (i.e. shear strength < 70kPa)
- No obstacles to depths of about 30 metres
- Relatively unrestricted overhead clearance
- Relatively vibration-free technology is required
- Treated ground strengths have to be closely engineered (typically 0.1 to 5 MPa).

The technique has the added advantage of very little spoil and heave being produced during installation, which eliminates the need to remove materials from site which is a costly practice.

2.2.3 Installation Patterns of Dry Deep Soil Mixed Columns

Dry deep mixed columns can be arranged in a number of ways in order to provide the required results. These patterns are dependent on the site conditions, cost of treatment, purpose of the columns and the quickest installation time, with either single or overlapping columns available.

Lime-cement columns are generally installed in single rectangular grid patterns, with diameters ranging from 500mm to 800mm and central spacings between 1.0 to 1.6 metres (Holm, 1999). However, overlapping arrangements have become increasingly common in Nordic countries. Figure 2.5 shows common patterns that can be adopted for deep soil mixing installations:

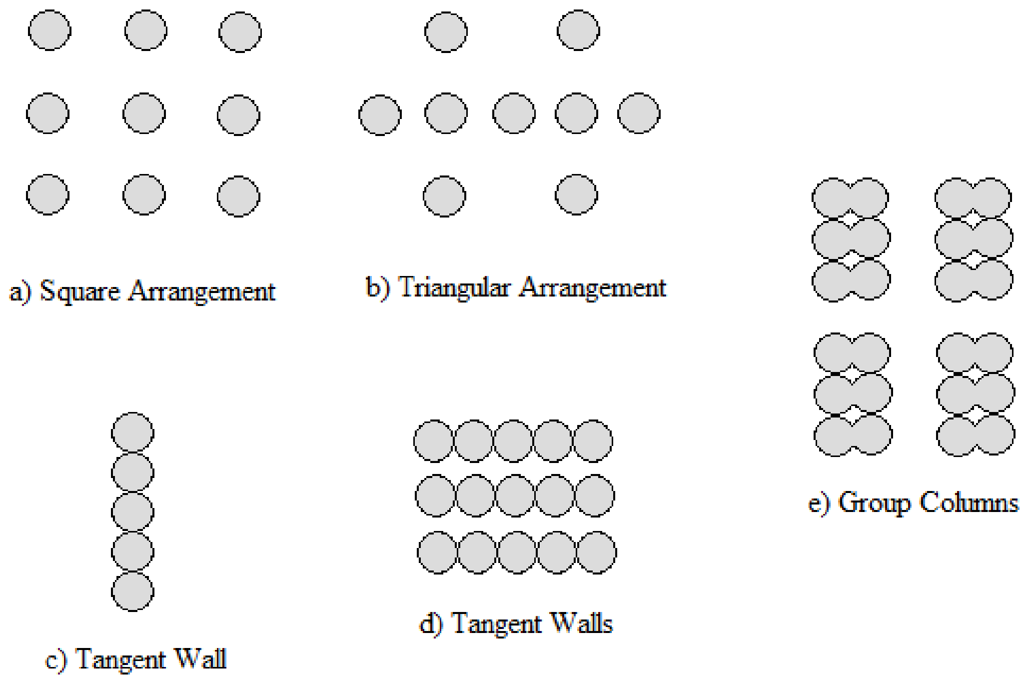


Figure 2.5: Typical Arrangement of Grouped Deep Mixed Columns (Topolnicki, 2004)

The square and triangular column type arrangements are usually applied to improve the stability of such applications as road and railway embankments. When the spacing between grouped columns does not exceed 1.5 to 2.0 metres then the load in a weak column can be transferred to an adjacent column with a higher bearing capacity (Fang, 1990).

2.2.4 Jet Grouting

Jet grouting is a deep *in situ* soil mixing technique which uses the energy of a high pressure jet of grout or water to disrupt the ground, causing modification and improvement in the process. In most common practices, a rotating drill rod is advanced to the required depth with high pressured water or grout being introduced as the rod is withdrawn. The high injection pressures cut and break the soil perpendicular to the angle of insertion, with the fragmented soil subsequently being mixed with the grout by the action of the rods rotation. This produces columns with a homogenously improved zone around the mechanically mixed core (Cementation Skanska, 2009).

Traditionally jet grouting systems are divided into three categories; single system, double system and triple system. The single system involves simply the injection of grout at high pressures; this process is known to provide the most homogenous column with the greatest value of strength. It also produces the least amount of spoil, which is beneficial as removal of waste from site is minimal (Earth Tech, 2010). The double system utilises both cement grout and air. By incorporating air, this system increases the efficiency with which the surrounding soil is cut and mixed and is generally used more in cohesive soils than the single system. The air reduces frictional losses incurred by the grout in the single system which allows the grout to travel farther from the injection point and hence facilitates larger diameter columns. The presence of air however results in a reduction of the column strength and uniformity throughout its length. Greater spoil is also brought to the surface which provides the disadvantage of added cost for waste removal. The triple system incorporates cement grout, air and water with the air and water being applied at greater pressures than the grout providing columns of greater quality than the other systems (Munfakh, 1997).

The main advantages of using jet grouting is the speed with which the columns are installed, the ability to apply the technique in locations of limited workspace and the ability of the technique to stabilise a variety of soil types. The main disadvantages to jet grouting are that the construction process can cause heave or excess lateral movements, with the high pressured grout causing fracturing to the surrounding ground. This can lead to problems if the installation is taking place in the vicinity of other buildings.

2.3 VIBRO DISPLACEMENT STONE COLUMNS

Vibro displacement stone columns are another recognised ground improvement technique developed in the 1960s for the purpose of reducing the overall settlement of soft cohesive soils. It was developed on the back of the vibro-compaction process, which had been successfully used to compact loose sand. Vibro-compaction relies on a vibrating poker device (vibroflot) to apply horizontal vibrations through the soil during penetration to the required depth (McCabe, 2007). These vibrations increase the relative density, and hence strength of the surrounding soil. However, the vibrations from the vibroflot device do not have any significant effect when applied to soft cohesive soils, such as clays and silts. For this reason, the penetration of the poker is accompanied by the construction of a vertical column of compacted gravel – referred to as a stone column. The vibro stone column process usually involves replacing 10% to 35% of the *in-situ* soil with crushed rock (McKelvey, 2004).

Stone columns have the potential to increase the bearing capacity of soft soils to between 150kPa to 400kPa depending on the column length, column spacing, column diameter, on the type of ground treated and whether the columns are situated on a composite high load bearing strata (Woodward, 2005). They have also been proven to reduce the overall settlement of soft soils (Wehr, 2004) by generating a drainage effect; which increases the settlement rate and provides a reduction to the consolidation time and compressibility. A rapid increase in the soils shear strength is also observed as excess pore water pressures (generated by some loading pressure) are reduced as a result of the columns drainage effect; drainage increases the effective stress experienced by the soil.

As the gravel column is a great deal stiffer than the soil; with vertical stresses in the column reported to be 2 to 5 fold higher than in the surrounding soil (TerraSystems Inc, 2010), a reduction in the total settlement of the system will occur during loading. Differential settlements are also reduced due to the ability of the rig controller to vary the diameter of stone columns along their length. This aims to combat different soil strength at varying depths and somewhat homogenises the stiffness properties of the soil/column matrix.

When loaded the stone columns transmit some load to the soil through the shear stresses, (operating along the column-soil interface) and some through end bearing (q_b). However, the main load-transfer mechanism involves lateral bulging (σ_r) of the column into the surrounding column (McCabe, 2007) as shown in Figure 2.6. Therefore the columns performance under loading and the performance of the ground between stone columns (i.e. the effective stiffness ratio), relies considerably on lateral support provided from the surrounding soil as well as the internal friction of the column (Woodward, 2005).

This lateral support is generated by the interaction of the column with the soil in the presence of a horizontal deformation (Wehr, 2004). Vibro stone columns are not suitable in liquid soils due to the small lateral support available to the column. For this reason stone columns have been limited to soils possessing shear strength greater than 15kPa (McKelvey, 2004). However, successful installation of columns has been achieved in soils with shear strengths between 5kPa and 15kPa (McCabe, 2009).

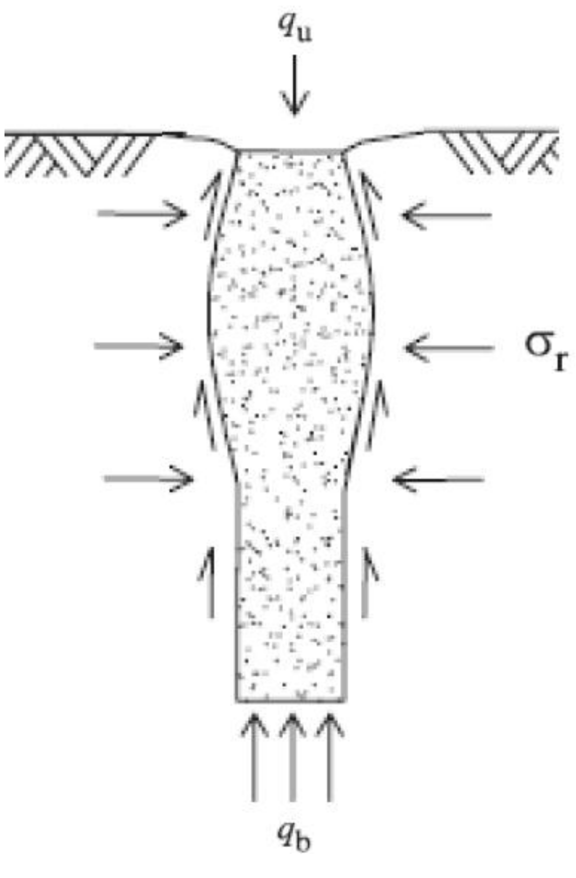


Figure 2.6: Load transfer mechanisms of Stone Column adapted from (McCabe, 2007)

Dry Top Feed Method

This is the simplest form of installing stone columns: with the vibroflot penetrating the soil through its own mass, the vibratory action of the poker and air flush in order to form a borehole. This method is predominantly used in shallow to medium treatment depths of coarse and more competent fine deposits. Once the required depth is reached the flot is removed, leaving behind a stable cavity hole which is then filled with stones. The flot is re-inserted and 'packs' the stone into the surrounding strata – 500-800mm successive charges of stone are added and compacted bringing the column up to working level (Balfour Beatty, 2010). This method is commonly used for lightly or heavily loaded developments, however is rarely suitable for soft cohesive soils (McCabe, 2009).

Dry Bottom Feed Method

The dry bottom feed method is predominantly used for treatment of water bearing and soft cohesive soils, where there is the likelihood of the borehole collapsing between float insertions during the top feed method. Again the vibroflot penetrates the ground using its own mass, vibration and air flush; however at design depth stones are introduced. Using a specially designed auger (refer to Figure 2.10) stones are placed into the bottom of the borehole, and similarly to the top feed method, the stones are compacted by repeatedly inserting and withdrawing the poker (refer to Figure 2.11).

2.3.1 Grouped Stone Column

The ability to meet absolute and differential settlement criteria usually influences the design of stone columns groups in soft soils ahead of bearing capacity (McCabe, 2009), i.e. settlement governs the length and adopted spacing between columns in grouped arrays. Figure 2.7 shows conservative design charts produced from Priebe (2000), which consider load distribution from the column and lateral support from the improved area surrounding the column to provide an improvement factor (n):

Where:

$$\text{Improvement Factor (n)} = \frac{\text{Settlement without treatment}}{\text{Settlement with treatment}}$$

The improvement factor (n) indicates the increase in compression modulus and the extent to which the settlement will be reduced as a result of the improvement caused by the column/soil interaction. It is a function of the angle of friction of the stones used as backfill and the Poisson's ratio of the soil (McCabe, 2007):

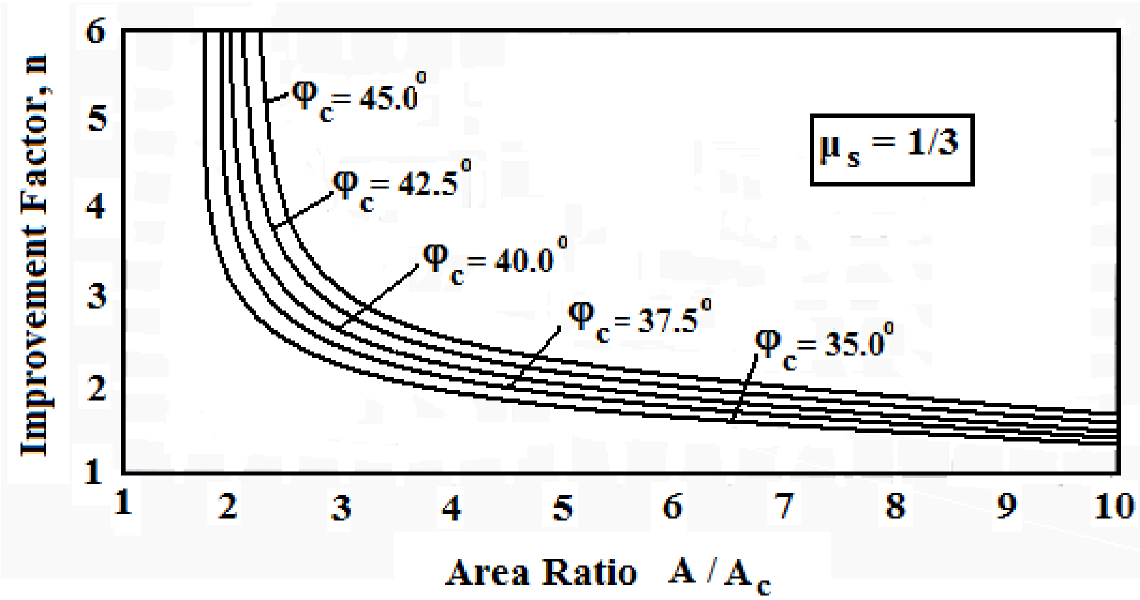


Figure 2.7: Priebe's chart for estimating post treatment settlement (Wehr, 2004)

The improvement factor (n) is also a function of the area replacement ratio which is dictated by the column spacing (Figure 2.8) and is found through A/A_c; where A_c is the cross-sectional area of one column and A is the total cross sectional area attributed to each column (McCabe, 2009). A/A_c is also dependent on the column spacing (s) and the column radius (r) to produce the following:

$$\frac{A}{A_c} = K\left(\frac{r}{s}\right)^2$$

Where k = 4/π and (2√3)/π for square and triangular column grids respectively. It has been reported (D. Muir Wood, 2000) that the area replacement ratio influences the extent of column interaction, with respect to load distribution between the columns and the surrounding soil (Figure 2.9). The research also claims that an area replacement ratio of 25% or greater is necessary for significant improvement in bearing capacity to be recorded.

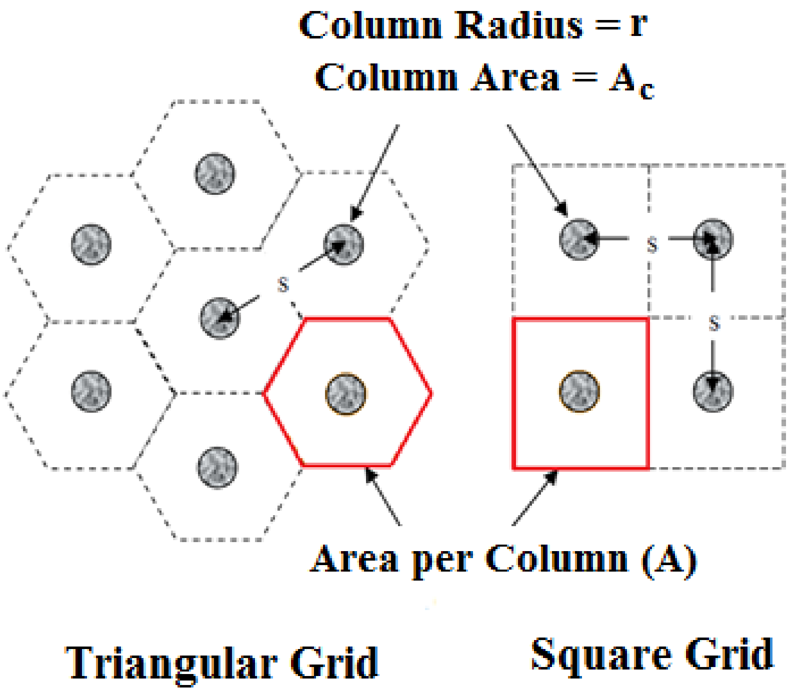


Figure 2.8: Typical arrangements of stone columns; adapted from (McCabe, 2007)

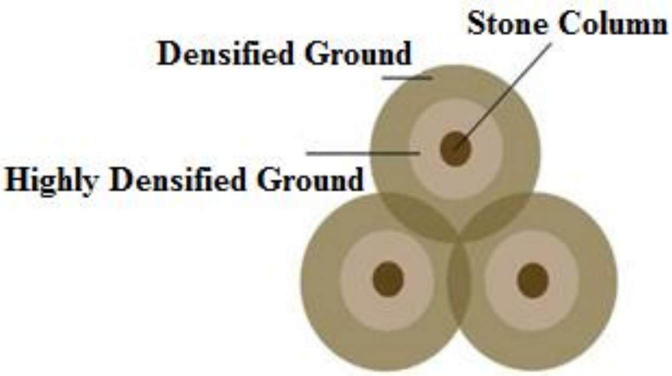


Figure 2.9: Possible zone improvement of grouped stone columns (McCabe, 2007)

The advantage of using stone columns for soil stabilisation over other techniques include the ability of stone columns to accelerate the dissipation of excess pore pressures generated during soil loading. This leads to quicker construction time, which coupled with the cheap cost of materials to produce the columns, has cost benefits for construction projects. Settlement can also be reduced by a factor of 1.5 to 3 times depending on the nature of soil being stabilised (Balfour Beatty, 2010).

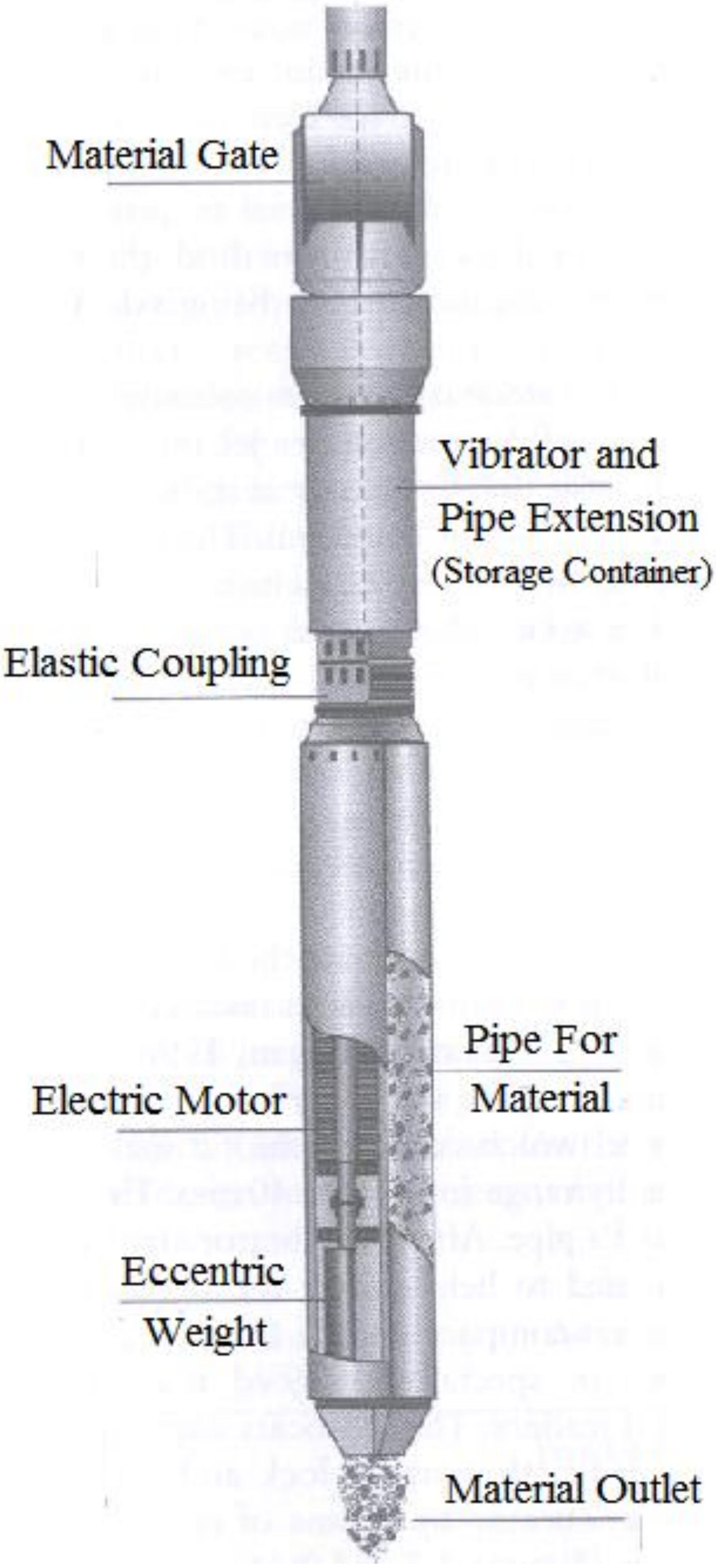


Figure 2.10: Schematic Detail of Bottom Feed Vibrator (Wehr, 2004)

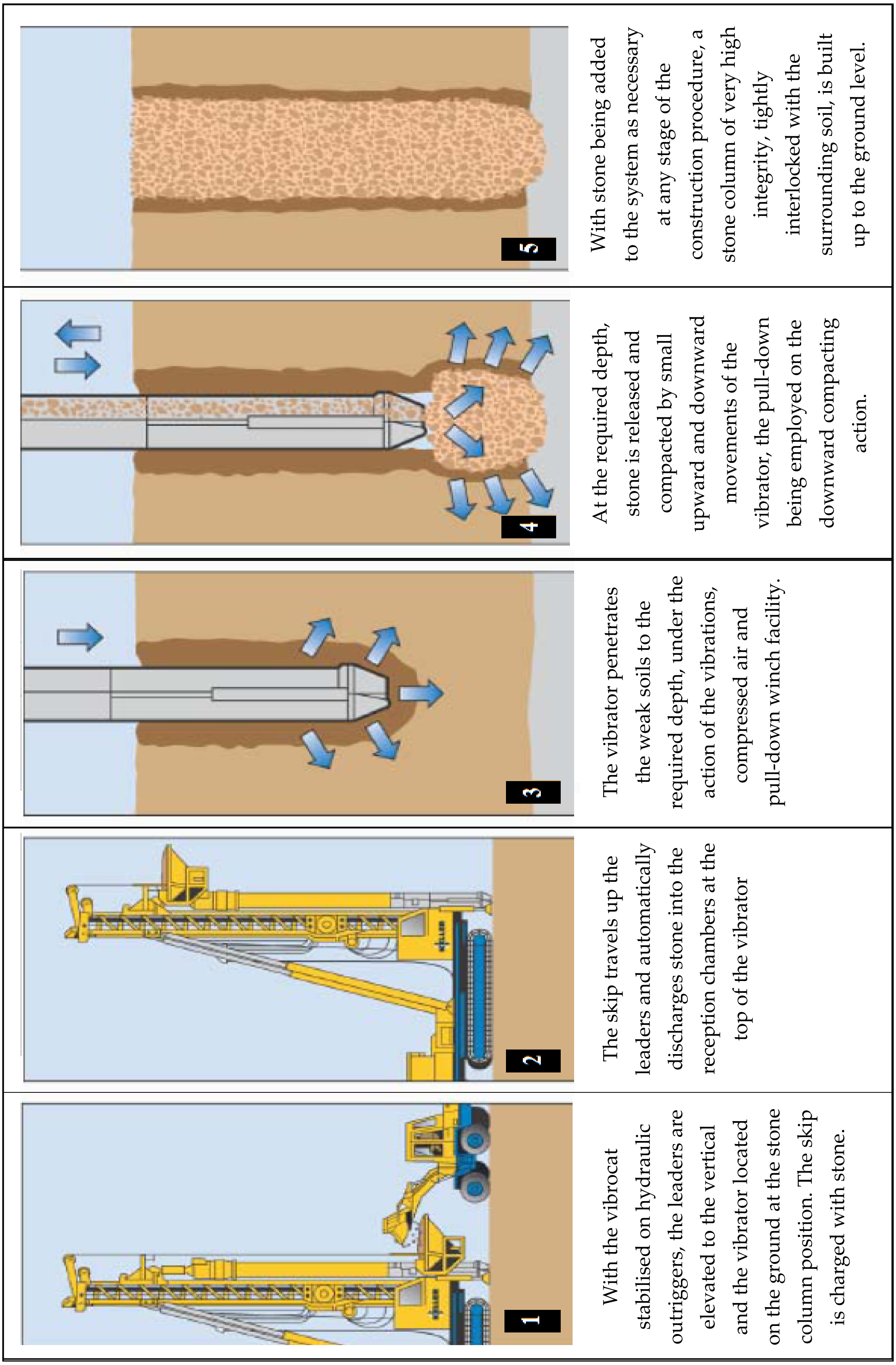


Figure 2.11: Bottom Feed Method of Stone Column Construction (McCabe, 2007)

2.4 CONTROLLED MODULUS COLUMNS (CMC)

Traditional practice dictates that when settlements are unacceptable, then deep foundations are required. However, this may not be the most cost efficient or economically friendly, sustainable technique available. Controlled Modulus Columns (CMC) are a relatively new ground improvement technique developed in France by Menard Ltd, who are part of the French Vinci Group, for the purpose of lightweight structures such as highways and railway embankments (Porbaha, 2006).

Although an established proprietary technique, there is little literature available to carry out an extensive review in terms of advantages over other ground improvement techniques. Private communications with a nationally known contractor (who have successfully implemented CMCs on a number of projects), has allowed the following literature to be provided which addresses the need for CMCs, the adopted installation process for CMCs and its advantages over other techniques.

The development of CMC technology was undertaken to overcome the difficulties experienced in controlling the modulus of a soil-cement column along its length (Miao, 2009). CMCs can generally be described as discrete, vertically installed semi – rigid mortar inclusions, which reach a predetermined stiffness ratio between the column and surrounding soil; creating a complex soil + inclusion composite material.

They are installed using a specially designed 3 part hollow-stem displacement auger (Figure 2.12), which is connected to equipment capable of delivering high torque capacity and high static down thrust. As the auger is driven to the required depth, the screwing motion of the bottom part of the auger causes the soil to be lifted into the vicinity of the middle section. This then displaces the soil laterally (DGI-Menard Inc, 2009) increasing the density of the surrounding soil and as a result increasing the soils load bearing capacity. Lateral movement also causes virtually no spoil to be brought to the surface, which reduces the need for removal of material off site. Also no vibrations are produced during installation; therefore the technique can be used in the vicinity of sensitive structures. Once the required depth has been reached the top section of the auger reverses its direction of rotation and the auger is extracted. This reversal in rotation ensures that the soil above the auger remains compacted. During extraction pre-mixed concrete slurry (with a controlled w/c ratio) is delivered under pressure which bonds to the surrounding soil providing column-type reinforcement. No soil mixing is performed at any stage of the installation/extraction process. Typical column diameter range from 350-500mm depending on the size of the auger.

Figure 2.12: Three Part CMC Displacement Auger (Menard, 2010)

(Barclay, 2009) indicates that enhanced construction time, improved quality control and reduced settlements are achievable when utilising CMCs compared to other improvement techniques. They also provide the advantage that the stiffening/strengthening of the soil reduces the need for staged construction of embankments, which reduces project costs. There are also no limitations in the ground conditions to which CMCs can be installed; unlike deep mixing techniques which are limited to soils with shear strength less than 70kPa. One limitation to the technique is that 25 metres is the maximum length of treatment from the soil surface.

The Author of this report was interested to know if any work had been undertaken incorporating dry cement to form CMCs, instead of applying pre-mixed slurry. The contractor had no knowledge of such a practice being carried out for CMCs and argued that a dry inclusion would be much more unpredictable in terms of *in situ* strength gain. This is due to the fact that a dry mix would be required to utilise water from the surrounding soil in order to hydrate, which is difficult to assess in the design process. Also dry cement would limit water ingress into the centre of the inclusion once sufficient hardening has taken place; thereby not all cement would participate in the hydration reaction and would be a waste of material.

Further theories behind CMC's adopting a wet mortar mix to install the columns, rather simply introducing dry cement, were expanded (Barclay,2009); firstly, the introduction of a pre-mixed mortar (with controlled w/c ratio) will result in more predictable *in situ* shear strengths, which is important for widely spaced discrete columns. This is because control cubes can be readily taken at the source which aids in predicting mix quality, the design strength and rate of design strength. Unconfined compressive strength tests are usually performed on the cubes at 7, 14 and 28 days, with a single CMC designed to support loadings of 150kN to 350kN.

Secondly, a wet mix can achieve a much higher degree of compaction which enhances the density/strength of final column. This can aid in situations where lateral forces could potentially cause damage to freshly grouted columns. The quality of execution of each CMC column is controlled by monitoring the speed of rotation and advancement of the auger, torque, down-thrust and drilling energy applied during installation. The pressure and volume of injected grout is carefully monitored and allows the column's stiffness to be constant along its length. The contractor argued that successful installation of dry columns is far more difficult to achieve than a wet mixture, as the slurry acts as a natural lubricant to the installation tubes. However in the Authors' opinion, this problem would not be too difficult to overcome as the Japanese DJM method is able to provide dry powdered binders through the installation tubes without any major problems.

Finally, the water added to produce the wet mix provides additional volume to the mix. As water is a low cost alternative to the expensive cement material this reduces the cost of installing each CMC column on site, whilst still providing all the benefits mentioned above.

Although these seem reasonable assumptions, the contractor was unable to provide any evidence that a dry inclusion would not be able to provide a further level of ground improvement, through soil dehydration, in comparison to that achieved by a wet mix.

2.4.1 CMC Grouping Patterns

CMC are typically installed in a square grid pattern with centre-to-centre spacing in the range of 1.2 to 3.0 metres, with a typical area replacement ratio of between 2% to 8% (as shown in Figure 2.13). As CMCs are installed using a displacement auger, which carries the risk of damaging the freshly grouted columns surrounding the installation of a new CMC, the columns must be installed in the following manner:

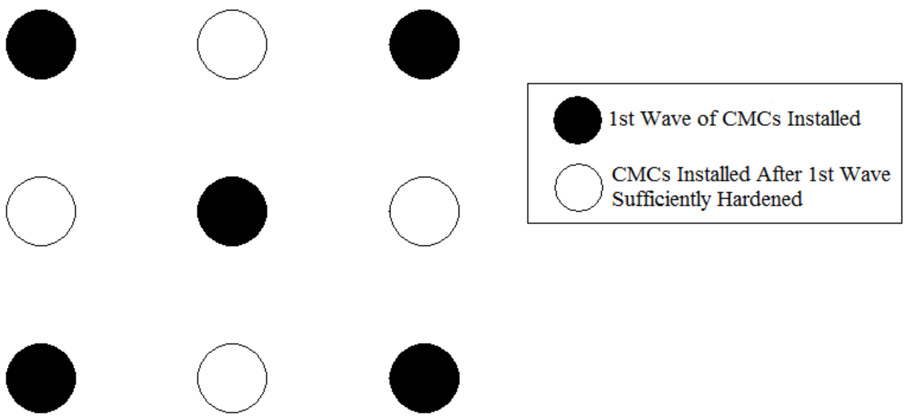


Figure 2.13: Square grid installation pattern for CMC Columns (Porbaha, 2006)

In the event of very soft ground conditions CMCs are installed in a densely spaced system with a much larger area replacement ratio.

2.5 DEWATERING EFFECT ON THE PROPERTIES OF A CLAY SOIL

It has been well established that reducing the pore pressure in a clay soil increases the soil strength by consolidation. The following section therefore only provides a brief description outlining the behaviour of a clay soil during consolidation and the key equations of the process.

When external forces such as buildings or embankments are constructed on the soil surface, the soil experiences a change in stress which can often lead to long term settlements and progressive softening of soil in excavations. Stress changes are a direct result of seepage flows, compression and swelling of soil as a result of these seepage flows and changes to effective stress; causing the soil to deform over time (Atkinson, 2007).

In order for a soil to deform the soil particles must change in volumetric packing, which can only be achieved with the expulsion of the pore water occupying the voids between the soil particles (i.e. a reduction in voids ratio). Equilibrium tells us that at all times the total external stress ($\Delta\sigma_v$) can only be supported by a combination of the effective stress between particles ($\Delta\sigma'_v$) and the pressure in the water ($\Delta\sigma'_v$) surrounding the particles (Atkinson, 2007):

$$\Delta\sigma_v = \Delta\sigma'_v + \Delta u$$

It has been well documented that clays are unable to rapidly respond to changes in stress as a result of their low permeability (Muir Wood, 2009). Therefore there is no immediate possibility of the soil particles experiencing a change in effective stress necessary to carry the applied load. As a result the pore pressure between the soil particles must increase in order to maintain equilibrium, i.e. the pore pressure carries the total applied stress whilst the effective stress remains unchanged. As there is no change in effective stress there is also no change in voids ratio.

Immediately above the ground surface no pressures exist. This pressure difference causes a hydraulic gradient to form and the water begins to flow to the region of lower pressure and drain at the free surface (Whitlow, 1995). As water is released from the soil the excess pore pressures begin to dissipate and the soil begins to settle, with the applied vertical load gradually transferring to the soil particles. As soil settles, the voids that exist between the solid soil particles reduce creating a more densely packed pore structure, this process continues until the total applied load is carried solely by the soil particles, which causes the strength of the soil to increase.

2.5.1 Normal Consolidation

Normally consolidated clay soils can be described as soils experiencing the highest effective stress ever applied within its history. Figure 2.14 shows how these soils tend to compress and reduce in voids ratio with increasing pressure:

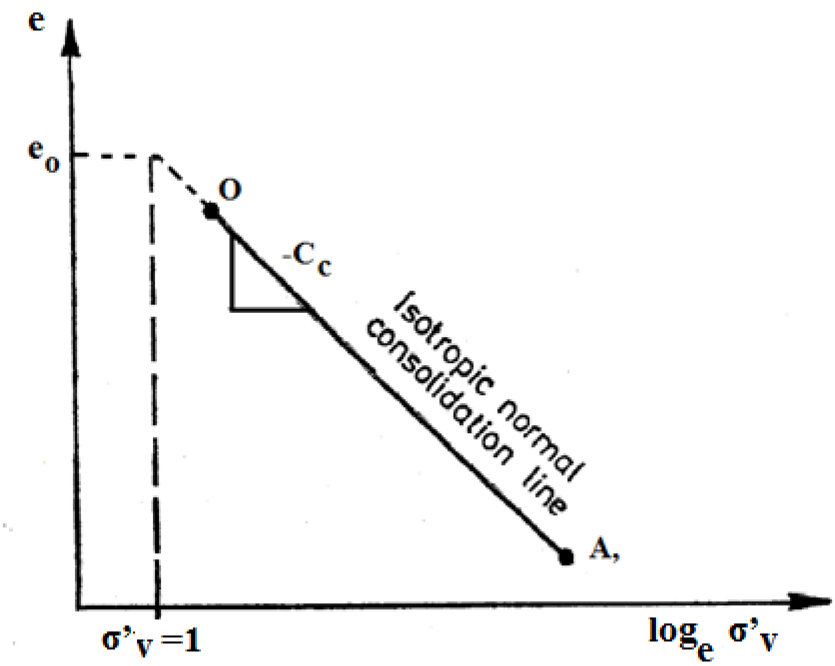


Figure 2.14: Normal Consolidation Line (NCL) adapted from (Ingles, 1987)

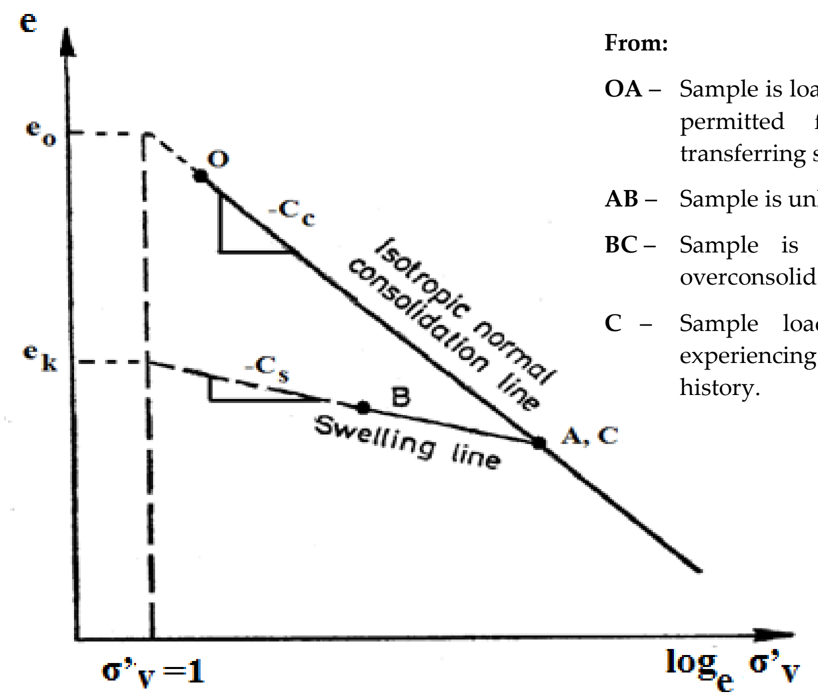
Providing the current stress is higher than the preconsolidation pressure this relationship is represented by the Normal Consolidation Line (NCL) which has the equation:

$$\mathbf{e = e_0 - C_c \log\sigma'_v}$$

Where: e_0 = is the voids ratio when the effective stress is equal to 1kPa, σ'_v = is the current effective stress acting on the soil and C_c = is the gradient of the normal consolidation line (NCL).

2.5.2 Overconsolidation

The behavioural characteristic (stiffness, strength, compressibility and specific volume) of clay soils is highly dependent on the loading history and on the current state of the soil (Atkinson, 2007). The overconsolidation ratio (OCR) is used to describe a soil which is currently experiencing a lower stress than the highest stress in the soils history. Figure 2.15 shows how these soils tend to swell, in the presence of water when unloaded and recompresses as the stress is increased shown by line ABC:



From:

OA – Sample is loaded; with expulsion of water permitted from the pore structure transferring stress to the soil.

AB – Sample is unloaded; relaxation of stresses.

BC – Sample is reloaded; but remains in overconsolidated state.

C – Sample loaded back onto NCL i.e. experiencing the largest stress in its history.

Figure 2.15 Swelling line (Ingles, 1987)

This swelling line is represented by the equation:

$$\mathbf{e = e_k - C_s \log\sigma'_v}$$

Where: C_s is the gradient of the overconsolidated (swelling) line.

As can be clearly seen in the above graph, once a soil has been loaded (line OA) and then subsequently unloaded (line ABC) the soil does not display elastic behaviour, as it moves away from the NCL. This is because the soil particle arrangement is more closely packed (dense) due to the soil skeleton resisting the external stress and the pore water being expelled from the voids between particles. As a result the structure of the soil particles has permanently deformed 'memory' and can only ever reach the NCL line if further pressure is applied equal to or beyond the preconsolidation stress.

The ratio of preconsolidation stress to the current mean effective stress is known as the overconsolidation ratio (OCR):

$$OCR = \frac{Preconsolidated\ Stress}{Current\ Mean\ Effective\ Stress}$$

It is the Authors belief that the ability of a dry binder to utilise pore water from a clay soil would results in a dewatering effect, causing the soil strength to increase, in a similar manner to the removal of pore water from the soil matrix which is facilitated in both the lime-cement column and stone column methods. As discussed in Chapter 1, the risk is that the removal of water from the soil will be limited by the absence of mechanical mixing as this is considered vital in the lime-cement column method.

2.6 CEMENT HYDRATION AND WATER INHIBITATION INTO CEMENTITIOUS SYSTEMS

The hydration process involves the reaction between cement and water, in which the silicates and aluminates of Portland cement form hardened products (Dhir, 1996). The main compounds can be broadly classified as tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF). The following section briefly describes the role of these during cements reaction with water.

Calcium Silicates

The hydration process is a very complex phenomenon which does not occur at a constant rate or even at a steadily changing rate. The two main calcium silicates; C_3S and C_2S, provide the bulk of unhydrated cement and it is their hydration products which provide the hardened cement paste with most of its engineering properties i.e. strength and stiffness (Neville, 2005).

On first contact with water, calcium hydroxide ions are rapidly released from the surface of each C_3S grain into solution. This release of ions causes the pH of the solution to increase and provides the conditions necessary for calcium hydroxide to crystallise and an 'outer' layer of calcium silicate hydrate (C-S-H) to form on the surface of the C_3S grain (approximately 10nm thick). As hydration continues the thickness of this layer increases and forms a barrier through which water must flow to reach unhydrated C_3S and through which ions must diffuse to reach the growing crystals. This impedes further hydration and initiates a dormant period; in which little reaction activity takes place. Eventually, movement through the C-S-H determines the rate of reaction and hydration becomes diffusion-controlled. The end of the dormant period is marked by a break in the barrier; as a result of either osmotic pressure or calcium hydroxide crystal growth and setting occurs as the hydrated products of each individual particle come into contact with one another (Black, 2006).

Tricalcium Aluminate (C_3A)

In its pure form C_3A reacts violently with water and can lead to a phenomenon known as *flash set*; where there is an immediate stiffening of the paste (Neville, 2005) due to the rapid formation of calcium aluminate hydrates. If this occurs then a porous C-S-H framework will form in which the remaining cement compounds hydrate, causing the strength characteristics of the cement paste to be adversely affected.

To prevent this occurrence gypsum is added to the cement clinker. In Portland cement the hydration of C_3A involves reactions with sulfate ions; which are supplied by the dissolution of gypsum forming calcium sulfoaluminate hydrate (or ettringite). The reaction of C_3A is strongly exothermic, but it does not last long. This is due to calcium hydroxide, liberated from the hydration of C_3S, reacting with C_3A and water to form C_4AH_{19}. This product creates a protective coating on the surface of the unhydrated C_3A grains, ultimately retarding C_3A hydration (Black, 2006); in a similar manner to the C-S-H during hydration of the calcium silicates.

The amount of gypsum added to the clinker has to be closely monitored, as ettringite is a stable hydration product providing there is an ample supply of sulfates available (Domone, 2001). The more gypsum there is in the clinker, the longer ettringite will remain stable. However, an excess of gypsum leads to unrestrained expansion; resulting in disruption to the paste microstructure. The optimum gypsum content is such that little reaction of C_3A is possible after all gypsum is exhausted, thereby providing a paste of lower pore sizes and consequently greater strength.

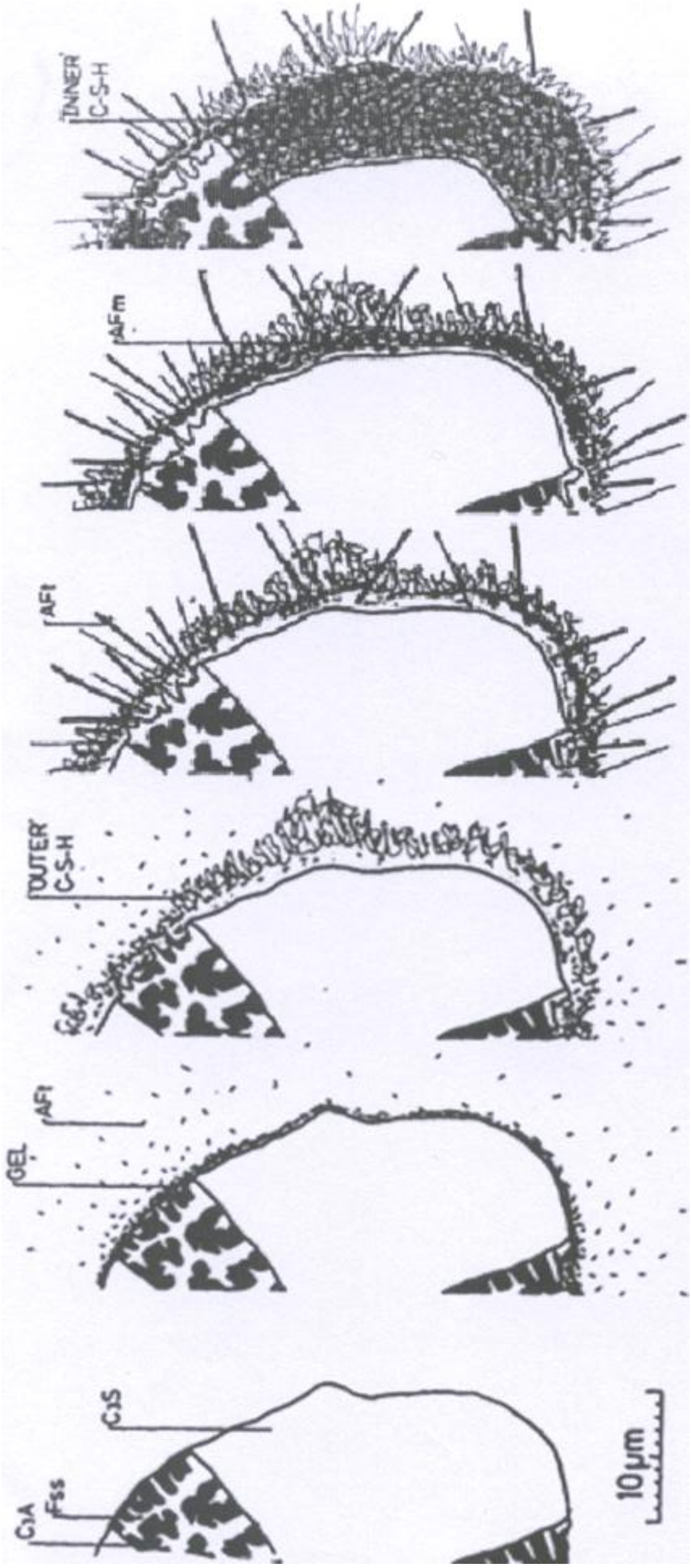
Each kilogram of cement has the potential to hydrate to some stoichiometric value, with grain size, fineness and chemical composition all influencing the level of hydration achieved. The potential volume of moisture reduction in a clay soil is likely to be 244ml for every 1kg of cement placed into an inclusion, based on the assumption that a water/cement ratio of 0.24 is necessary for hydration to occur (ACI Committee 302, 2006). However, this theoretical value is argued to be too high an approximation as cement never reaches complete hydration and the core of cement grains never hydrates. In reality it is more likely that a value of 60% to 70% of this 244ml will be absorbed.

2.6.1 Predicting Water Movement into Dry Cement

Models to describe the way in which water moves into dry cement and initiate a hydration reaction have been well established, with Taylor's model (refer to Figure 2.16) and its derivatives the most widely accepted (Grutzeck, 1999). However, these models cannot be considered relative to this project of work, as they only consider water being admixed throughout the cementious material; will not occur in the system proposed in this study. It is therefore likely that a different mode of water transportation into the cement will result.

However, there appears to be no literature available detailing such a process or the level of absorption likely to be encountered using this technique. For further confirmation of the lack of literature available a patent attorney was contracted to provide a comprehensive search. No model or systems similar to the one proposed in this study were found; which suggests that it has not been done.

The mechanisms by which water moves through cement have been well established and can be reviewed in a range of textbooks. A brief description of each mechanism and the important equations which accompany each mechanism is provided in the following literature. However, it should be noted that these mechanisms are valid only for hydrated materials.



a) Unhydrated Section of poly-mineralic grain (Scale exaggerated)

b) 10 minutes – Some C3A reacts with calcium sulphate in the solution. Amorphous Aluminate-rich gel forms on the surface and short ettringite rods nucleate at edge of gel and solution

c) 10 Hours – Reaction of C_3S to produce 'outer' product C-S-H and ettringite rod network leaving 1μm between grain surface and hydrated shell

d) 18 Hours – Secondary hydration of C_3A producing long rods of ettringite. C-S-H 'inner' product starts to form on inside of shell from continuing hydration of C_3S

e) 1 to 3 Days – C_3A reacts with any ettringite inside shell forming hexagonal plates of monosulphate. Continuing formation of 'inner' product reduces separation of anhydrous grain and hydrated shell

f) 14 Days – Sufficient 'inner' C-S-H has formed to fill in the space between grain and shell. The 'outer' C-S-H has become more fibrous

Figure 2.16: Portland cements microstructural development during early age hydration (Taylor, 2004)

2.6.2 D'Arcy's Law

Flow in capillary pores follows the principles of D'Arcy's Law for laminar flow through a porous medium. However, D'Arcy's Law has been generalised to apply to any fluid flowing in any direction through a porous material as long as the flow is viscous (Basheer, 2001):

$$v = \frac{Q}{A} = -(\frac{K'\rho g}{\eta}) \cdot (\frac{dP}{dL})$$

Where: K' = Intrinsic permeability of the porous medium, dP = pressure loss over the flow path; dL (metres), η = dynamic viscosity of the fluid in N s/m^2, Q = flow rate, m^3/s, A = cross sectional area (m^2), v = apparent velocity of flow, ρ = density of the fluid (kg/m^3) and g = acceleration due to gravity.

Intrinsic permeability is considered the most rational concept of permeability, as only the characteristics of the porous medium, and not the fluid involved, is considered (Neville, 2005). As the fluid involved is generally water, the following can be produced:

$$K = \frac{K^{'}\rho g}{\eta}$$

The coefficient K is expressed as meters per second and is referred to as the coefficient of permeability.

2.6.3 Sorptivity

Capillary suction (sorptivity) is the process by which water or liquid state materials are absorbed by a porous medium. The rate at which adsorption occurs is a function of capillary pore size, interconnection of capillary pores and the moisture gradient. If the capillary is either dry or slightly wet, an immediate suction of fluid into the concrete will occur if any contact is made with a solution. (Verghese, 1991) states: "The depth of capillary suction is inversely proportional to the diameter of the capillary pore on the assumption that the capillary is continuous."

Studies carried out by (Hall, 1989) shows that sorptivity can be determined through the following equation:

$$i = st^{0.5}$$

Where: i = increase in mass per unit cross-sectional area in contact with the water, divided by the density of water (expressed in mm), s = sorptivity, expressed in $mm/mm^{0.5}$ (To express in SI units: 1 $mm/mm^{0.5}$ = 1.29×10^{-4} $m/s^{0.5}$) and t = time at which mass is determined (expressed in minutes).

2.6.4 Diffusion

Diffusion is the process by which matter is transported from one part of a system to another due to a concentration gradient. The movement occurs as a result of small random molecular motions, which take place over small distances. Fick's First Law of Diffusion states that the rate of transfer of mass through unit area of a section, J, is proportional to the concentration gradient dc/dL (Baker, 2004). This can be expressed as:

$$J = -D\frac{dc}{dL}$$

Where: J = mass transportation rate in kg/m^2 s, D = effective diffusion coefficient in m^2/s, L = thickness of the sample in meters, $\frac{dc}{dL}$ = concentration gradient in kg/m^4.

As mentioned these mechanisms are only valid for hydrated materials. The mechanism by which water will transfer through the cement inclusions is at this stage unknown, as there appears to be virtually no literature in which water transfers through dry cement without the aid of admixing water throughout the cement.

2.7 PREVIOUS RESEARCH CONCERNING THE APPLICATION OF DRY CEMENT INCLUSIONS

The aim of this study is to investigate the strength improvement of a clay soil, after an inclusion consisting of dry, unhydrated Portland cement has been successfully installed. The intention is for the dry cement to utilise pore water from within the clay soil, in order to hydrate, causing a dewatering effect to be experienced in the soil surrounding the inclusion. However, as detailed in Chapter 1 the risk is that the cement effectively seals the core of the inclusion to the water available in the soil once a certain degree of hydration and hardening has ensued; thus limiting the cements interaction with the pore water.

This method of stabilisation is not a new concept. The following literature details previous investigations utilising dry cement inclusions in order to dewater the moisture within the surrounding soil. However, these investigations differ from this current research as different experimental techniques, soil material and ground conditions were under investigation.

One example of this type of improvement system was presented by Duraisamy (2007) where dry cement columns were used to stabilise Tropical Peat. This study focused on the improvement of the compressibility parameters (C_c and C_α) of tropical peat where dry cement columns were installed prior to consolidation in a standard Rowe Cell. Duraisamy investigated two column diameter of 45mm (Area Replacement Ratio = 0.09) and 60mm (Area Replacement Ratio = 0.16), which were cured for 7, 14 and 28 days respectively. The results obtained showed that the larger column diameter had a greater influence on reducing the compressibility parameters, with the samples cured to 28 days being the most effective. Duraisamy also performed Rowe Cell tests on groups of dry columns and found that the larger the Area Replacement Ratio (i.e. the larger and greater the number of columns) the greater the improvement of the soil. Once again the curing period had a significant influence with 28 day cured samples showing the most improvement.

This dry column technique was once again studied by the same Author in Duraisamy (2009), with tropical peat simply being replaced by fibrous peat. The exact same conclusions were drawn, i.e. a greater number of cement columns, cured for longer periods, produced the most significant improvement.

These two studies performed show there is potential for dry cement columns to stabilise and strengthen weak soils. However, as these studies involved peat, which resembles no similarilties to clay, it is not a fair assessment of how the dry columns will perform in clay soils at different moisture contents.

To some extent, Larsson (2005) studied the possibility of introducing inclusions of dry cement into soil, when assessing the multiple factors which influence installation of lime-cement columns. One test programme adopted by Larsson, which is of particular interest to this investigation, focused primarily on the influence of the shape of mixing tool and number of mixing blades. Within this programme, 32 columns consisting of dry cement binder were installed, with no form of mixing taking place between the binder and surrounding soil, using the equipment shown in Figure 2.17. The inclusions were installed in soil which varied between 38 and 70% moisture content.

Figure 2.17: Mixing tool with no blades adopted in (Larsson, 2005)

The results show that out of the 32 dry inclusions installed, 14 still consisted of dry binder at the central core of the inclusion, with the remaining inclusions described as stiff with a grainy consistency. This would seem to suggest that the incorporation of a dry cement powder into a cavity, without any form of mixing taking place, is not a reliable solution for ground stabilisation. This can be said as 44% of the inclusions in the investigation were shown not to have any interaction with pore water from the surrounding soil near the central core of the inclusion, i.e. the system seems to shut itself off from further water ingress once the outer layers of the inclusion sufficiently hardened.

However, certain practices adopted by Larsson could have had a significant influence on the ability of cement to interact with the pore water. Firstly, the length of curing time afforded to the inclusions is questionable, as the inclusions were left to cure for a period of only seven to eight days after installation. Common practices adopted for testing concrete cubes involve testing at 3, 7, 14 and 28 days, as curing time has a significant influence on the interaction between cement and water. By limiting tests to 7 days, the possibility of these inclusions allowing water to continually migrate to the central core of the inclusion after this time is removed.

Secondly, the lack of a controlled inclusion diameter could have influenced the inclusions performance. The inclusions diameter were reported to vary between 100mm and 200mm, however Larsson provides no indication of which size of inclusions maintained dry powder at the central section. This would have been useful information at it might have suggested a limitation in inclusion diameter at which full penetration of water to the central section of the inclusion is achievable, i.e. all inclusions in the region of 100mm could have hydrated at the centre, with larger inclusions showing dry powder still in the centre. As this is not provided it is impossible to say.

From Larsson and Duraisamy's research, what is easy to identify is that cement does have the ability to absorb pore water from the surrounding soil without the aid of mixing. However, it is possibly the soil conditions, length of curing time and inclusion diameter which dictates the performance of dry cement inclusions and their ability to utilise moisture from the soil and increase the soil strength.

2.8 CHAPTER SUMMARY

The following points can best summarise the literature reviewed in this Chapter:

i. As has been well established, the strength of a soil increases as a result of a reduction in moisture and its behaviour can be described by the theory of consolidation.

ii. Lime-cement columns dehydrate the surrounding soil through hydration, ion exchange, flocculation and pozzolanic reaction, and can increase the compressive strength by three to four times in comparison to the untreated soil. The Japanese DJM relies on the hydration of the cement with the surrounding soil.

iii. Limitations to the current mixing tools and equipment used during installation have restricted lime-cement columns to soils with a shear strength < 25kPa and Japanese DJM to soils < 75kPa.

iv. Utilising dry cement, in place of a slurry mix, can increase the magnitude of strength improvement by a factor of 1.7 to 3.2 due to the reduction of moisture content; with the undrained shear strength increasing with an increase in cement content.

v. Based on the knowledge that a water/cement ratio of 0.24 is necessary for hydration to occur, it may be to possible to reduce the soils water by up to 244ml of water with every 1kg of cement used in a inclusion. However, in reality it can be expected to be that about 60% to 70% of this value will actually be removed for cement hydration.

vi. Mechanical mixing is argued to be essential in obtaining a homogenous material, with all cement particles participating in the hydration reaction.

vii. Increasing the cement content is problematic in the deep mixing techniques as more mixing is required to obtain a homogenous mix; this increases the energy requirements for installing the column. An increase in cement content also reduced the permeability of the column with increased curing time, thereby prohibiting the penetration of water through the columns cross section, i.e. limits continual drainage of the surrounding soil.

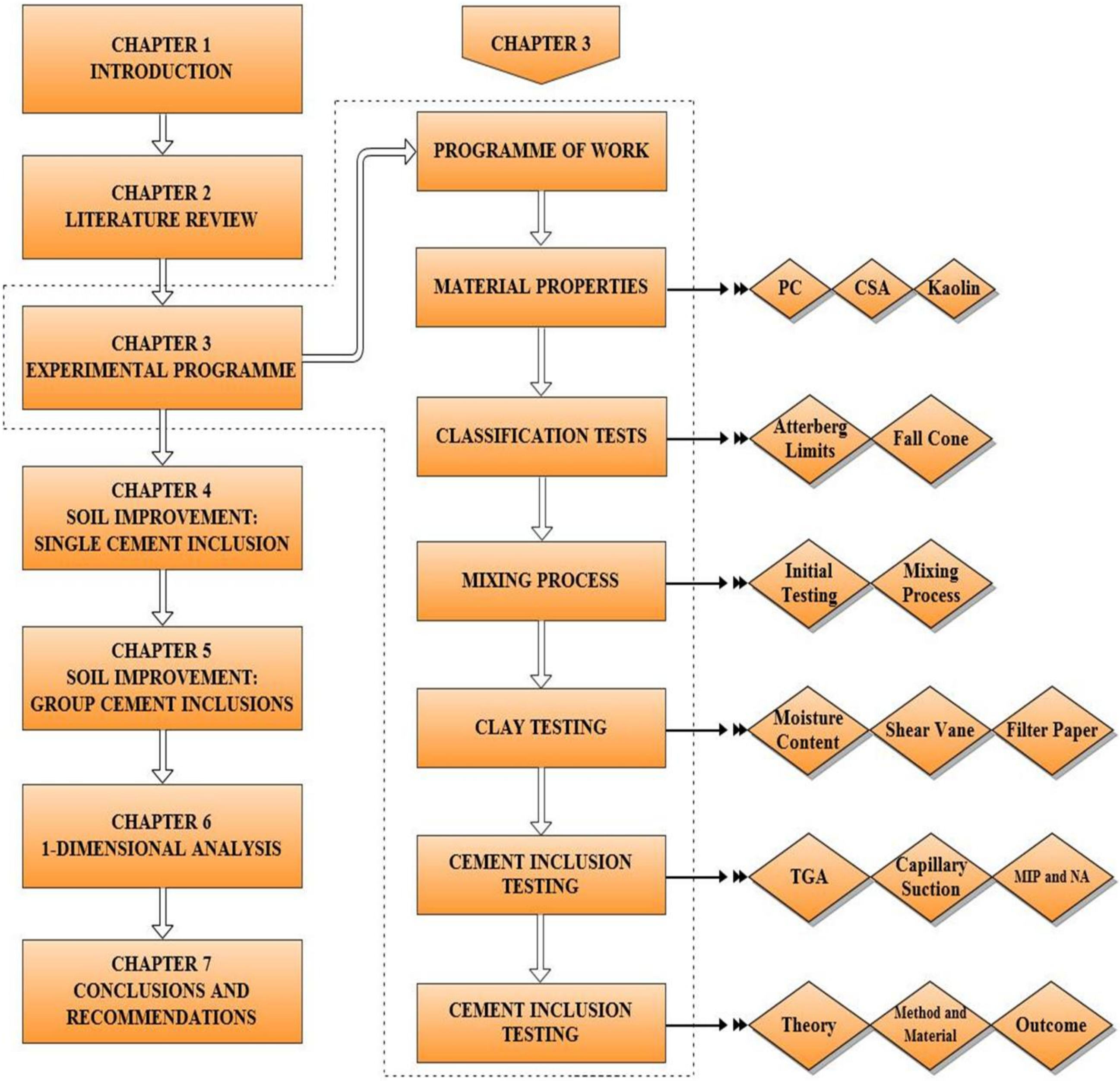
viii. CMCs use a 3-part hollow stemmed auger; which laterally displaces the surrounding soil to enhance the density of the soil. During auger extraction, ready mixed concrete slurry is placed into the cavity which creates a pre-determined stiffness ratio between the column and surrounding soil.

ix. Stone Columns have been known to reduce settlements by a factor of 1.5 to 3.0 times depending on the nature of soil being stabilised.

x. Dry deep mixed methods can adopt a wide variety of single or overlapping column group arrangements depending on the nature of the soil and the level of improvement required from treatment; common arrangements are square and triangular grids. CMC grouped columns adopt a square on square grid with an area replacement ratio of 2% to 8%.

xi. The technique proposed in this programme of work; installing dry cement inclusions to dehydrate the soil and increase its strength, has been used in two previous investigations involving peat soil and Rowe consolidation tests. Findings from these investigations concluded that installing inclusions of dry cement can significantly increase the soils strength, with increased curing time and area replacement ratios having a positive influence on the scale of improvement.

xii. Larsson (2005) provides an argument that installing dry cement into an excavated hole, without the operation of mechanical mixing, will not always allow water to interact with the cement at the core of the inclusion. However certain practices adopted by Larsson could be argued to have a negative influence on the ability of the inclusions to utilise pore water.



> "A good engineer thinks in reverse and asks himself about the stylistic consequences of the components and system he proposes"
>
> **Helmut Jahn (1940-Present)**

CHAPTER 3

EXPERIMENTAL PROGRAMME

3.1 EXPERIMENTAL PROGRAMME

The main aim of this present study was to examine the dewatering effect experienced by a clay soil, of known moisture content, when an inclusion of dry unhydrated cement was introduced into the soil. This section will detail the materials, equipment, test standards and procedures adopted by the Author in order to achieve the objectives stated in Chapter 1.

The experimental programme is divided into two parts, the first part includes details of the preparation and tests conducted on the clay soil and the second part discusses tests performed on the hardened cement inclusions. The clay soil was tested for strength whilst the hardened cement inclusions were tested for hydration.

Figures 3.1 and 3.2 summarise the overall research and experimental programme for this study, whilst Tables 3.1 and 3.2 provide an overall summary of which tests were performed and at what stages after installing the inclusions. (Note: in Table 3.1 and Table 3.2, the symbol (✓) highlights what conditions have been applied during testing).

3.2 TEST MATERIALS

3.2.1 Portland cement

Portland cement is the most widely used construction material in modern construction and is used in a number of the established techniques to stabilise soft clay soils. Portland cement, to BS EN 197: CEM 1, will be used in this investigation to produce the dry inclusions. Most physical properties have been provided by the suppliers (refer to Table 3.3); with the fineness measured by the Author using standard Blaine tests (BS EN 196-6: 1992). The chemical properties of the cement were analysed using XRF (refer to Table 3.4).

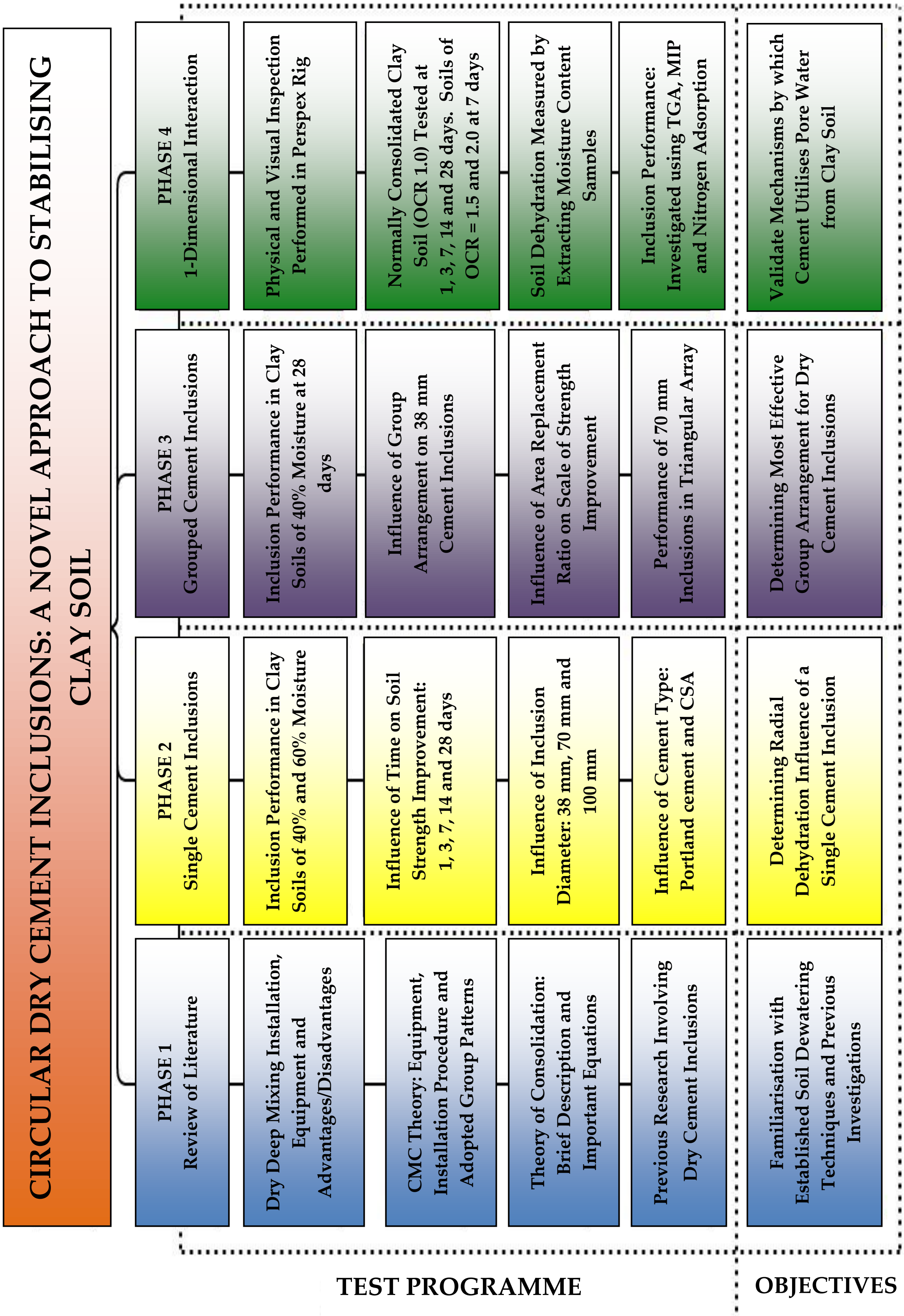


Figure 3.1: Overall Research Programme of Work

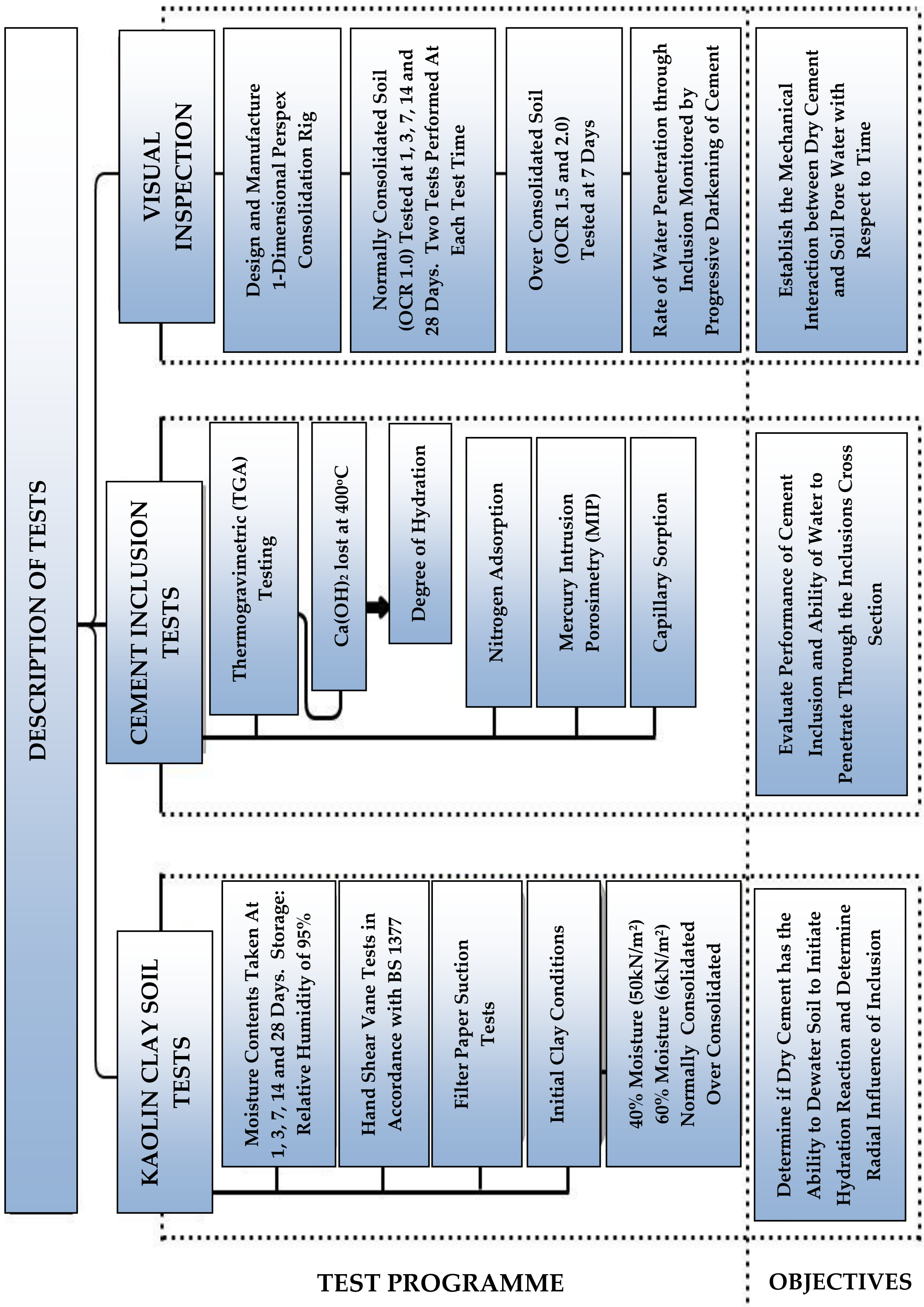


Figure 3.2: Experimental Programme of Work

Table 3.1: Summary of experiments performed for singular and grouped inclusions

SOIL MOISTURE CONTENT (%)	ARRANGEMENT OF INCLUSIONS				INCLUSION CEMENT TYPE	INCLUSION DIAMETER (mm)	TEST PERFORMED:		TEST PERFORMED (DAYS) AFTER INSTALLATION				
	SINGLE	SQUARE GRID	TRIANGLE GRID	CIRCLE GRID			MOISTURE CONTENT/ SHEAR VANE	TGA	1	3	7	14	28
60	✓				*Portland cement*	*38*	✓	✓	✓	✓	✓	✓	✓
40	✓				*Portland cement*	*38*	✓	✓	✓	✓	✓	✓	✓
40	✓				*Portland cement*	*70*	✓	✓	✓	✓	✓	✓	✓
40	✓				*Portland cement*	*100*	✓	✓					✓
40	✓				*Calcium Sulfoaluminate*	*38*	✓	✓	✓				
40		✓			*Portland cement*	*38*	✓	✓					✓
40			✓		*Portland cement*	*38*	✓	✓					✓
40				✓	*Portland cement*	*38*	✓	✓					✓
40			✓		*Portland cement*	*70*	✓	✓					✓

Table 3.2: Summary of 1-dimensional experiments performed in rig

SOIL CONDITION	TESTS PERFORMED: SAMPLE 1		TESTS PERFORMED: SAMPLE 2			TEST PERFORMED (DAYS) AFTER INSTALLATION:					TOTAL NUMBER OF TESTS	VISUAL INSPECTIONS PERFORMED AT (MINUTES):						
	WATER CONTENT / SHEAR VANE	TGA	SORPTION	MIP	NITROGEN	1	3	7	14	28		1	5	10	30	60	120	180
NORMALLY CONSOLIDATED (OCR 1.0) SOIL	✓	✓				✓	✓	✓	✓	✓	*10*	✓	✓	✓	✓	✓	✓	✓
NORMALLY CONSOLIDATED (OCR 1.0) SOIL			✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓
OVER CONSOLIDATED (OCR 1.5) SOIL	✓	✓						✓			*1*	✓	✓	✓	✓	✓	✓	✓
OVER CONSOLIDATED (OCR 2.0) SOIL	✓	✓						✓			*1*	✓	✓	✓	✓	✓	✓	✓

Table 3.3: Physical properties of Portland cement

Property	Value
Fineness, m^2/kg	340
Particle Density, kg/m^3	2350
Specific Gravity	3.1
Mean Particle Size, µm	20
Particle Shape	Angular

Table 3.4: Chemical properties of Portland cement

Parameters	
Chemical Characteristics, Oxide Analysis, weight ratio (%)	
SiO_2	20.0
Al_2O_3	5.0
Fe_2O_3	3.0
CaO	65.0
MgO	1.1
SO_3	2.4
Na_2O	0.2
K_2O	0.9

3.2.2 Calcium Sulfoaluminate Cement (CSA)

CSA is fast setting cement capable of producing increased early strength gain in comparison to traditional Portland cement. It is formed in a cement rotary kiln between temperatures of 1000°C and 1300°C, with the main mineral components in the clinker composed of anhydrous C_4A_3S (tetracalcium trialuminate sulfate or 'yeelimite'), belite (C_2S), Al-rich 'ferrite' and gypsum (Ambroise, 2004).

It is the reactive phase C_4A_3S which contributes to the early strength gain of the cement as it hydrates to form ettringite ($C_6A\bar{S}_3H_{32}$) as shown in the equation (Gastaldi, 2009):

$$C_4A_3\bar{S} + 2C\bar{S}H_2 + 34H \rightarrow C_6A\bar{S}_3H_{32} + 2AH_3$$

The advantages of using CSA over traditional Portland cements, apart from increased early strength gain, are best reflected when considering CO_2 emissions. Approximately 0.21 tonnes of CO_2 is released during the production of 1 tonne of CSA cement (Quillin, 2007). This is an energy saving of approximately 22%, in comparison to the 0.97 tonnes of CO_2 released for every tonne of Portland cement produced (McLeod, 2005). This reduction is a direct result of the CSA clinker requiring a lower proportion of limestone in the raw feed and being manufactured at lower operational temperatures in comparison to traditional Portland cement clinkers.

A comparison between the CSA and traditional Portland cements performance to stabilise clay will be made. Again, the typical physical properties of CSA were provided from the supplier (Table 3.5), with the chemical properties (Table 3.6) also given.

Table 3.5: Physical properties of CSA

Property	Value
Particle Density, kg/m^3	1000-1200
Mean Particle Size, μm	5 - 30
Particle Shape	Particulate

Table 3.6: Chemical properties of CSA

Mainly a mixture of the following chemical compounds, weight ratio (%)	
$CaO\text{-}3Al_2O_3\text{-}CaSO_4$	71 - 75
$4CaO\text{-}Al_2O_3\text{-}Fe_2O_3$	4 - 7
$2CaO\text{-}SiO_2$	14 - 18
$CaO\text{-}TiO_2$	2 - 4
With small amounts of:	
$CaO\text{-}TiO_2$	
$CaO\text{-}Al_2O_3$	
$2CaO\text{-}Al_2O_3\text{-}SiO_2$	

3.2.3 Kaolin

Kaolin was used to represent *in situ* clay soil for the purposes of this investigation, because of its high plasticity (allows material to be easily moulded) and ability to be remain stable when mixed to different moisture contents. It is also locally sourced and therefore readily available.

The material was supplied in white powdered form by the supplier; therefore the addition of water was necessary to obtain a material representative of *in situ* clay soils. The general properties of the kaolin have been provided by the supplier and are provided in Table 3.7.

Table 3.7: Kaolin properties

Chemical Analysis by XRF	
SiO_2 (mass %)	47
Al_2O_3 (mass %)	38

The mineralogy of the kaolin contains finely divided silicaceous and aluminous products that will form the cemented products of (C-S-H) and (C-A-H) in the presence of water and calcium hydroxide. 85% of the China Clay mineralogy consists of Silicon Dioxide (SiO_2) and Aluminium Oxide (Al_2O_3) with the remaining 25% consisting of mica, quartz and feldspar or illmenite by mass. This is beneficial for this study as this will allow a pozzolanic reaction to take place in the presence of Portland cement and hence stabilisation can occur.

3.2.4 Inclusion Diameters and Equipment Selected For Testing

For the purposes of this research, an inclusion is defined as an intentional placement of dry cement into a cavity hole, which is free to absorb and react with the soils pore water in order to hydrate; strengthening the surrounding soil as a result.

To form the circular shape of the inclusion within the clay soil, hollow steel pieces were used (refer to Figure 3.3). The inclusion diameters adopted for this study were arbitrary values of 38mm, 70mm and 100mm.

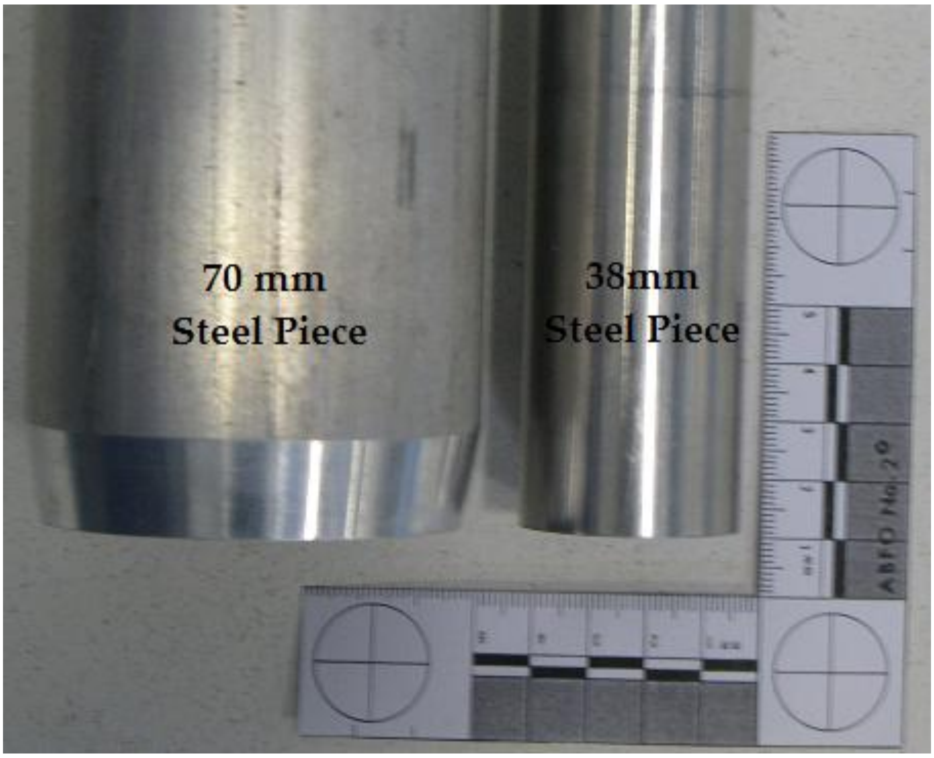


Figure 3.3: Hollow circular steel pieces used to form cement inclusion in clay

The 38mm diameter was initially selected as this could potentially be tested in a triaxial cell to determine the strength of the hardened inclusion. With potential strength tests in mind, the length of the inclusions were two times the size of the inclusion diameter (i.e. slenderness ratio of 2), which is standard practice when performing compressive strength tests on cylindrical samples.

The 70mm and 100mm diameters were tested to determine if increasing the size of the inclusion could provide a greater radial influence to the surrounding soil; as a result of an increase in cement content. The size of the container selected for analysing the single 38mm inclusions was 275 x 200 x 170 mm in size, with the inclusion positioned in the centre of the container. This allowed approximately 120mm space between the inclusion and the edges of the container for moisture samples to be extracted. The depth was sufficient for an inclusion length of 75mm.

For singular 70mm inclusions, a container with dimensions 390 x 300 x 200 mm was adopted. Again this provided ample space for between the inclusion and container edges, with sufficient depth for an inclusion length of 140mm. The 100mm inclusion was tested in a container 500 x 380 x 190 mm in size. Unfortunately, the depth of the 100mm sample was unable to reach a length of 200mm (to maintain a ratio of 2:1), therefore was placed to a depth of 140mm, similar to the 70mm inclusions.

3.3 CLAY CLASSIFICATION TESTS

Before beginning any experimental work it was important to classify the clay soil in order to identify its characteristics and mechanical properties. The following section details the classification techniques that were adopted for this study.

The Atterberg Limits test is a primary form of classification for a cohesive soil which relates the behaviour of the soil to its moisture content. These tests were carried out in accordance to BS 1377-Part 2: 1990, which allows the consistency limits for the soil to be determined.

The plastic limit is defined as the moisture content at which the soil becomes so brittle that it crumbles i.e. the transition point between the plastic and semisolid states.

The liquid limit is defined as the moisture content at which the soil begins to flow like a liquid i.e. the boundary between plastic and viscous flow states (Atkinson, 2007). The liquid limit test was performed using a standard fall cone.

3.3.1 Fall Cone Test

This test was used to determine both the liquid limit of the kaolin clay and the strength of the clay at different moisture contents. It is a quick and consistent technique which provides an indication of the soil strength by the depth of the indentation left by a cone dropped with a known force (mg).

An 80g cone, with a tip angle of 30° was allowed to fall under its own weight into the prepared clay sample, in such a manner to cause rapid penetration of the cone into the level clay surface (Figure 3.4). The depth of penetration was recorded 5 seconds after releasing the cone, with depth measured to the nearest 0.1mm.

From this data it was possible to determine the shear strength of the clay, using the equation proposed in (Muir Wood, 2009):

$$\frac{C_u d^2}{mg} = f(\omega) = k_\omega$$

W**here:** C_u = undrained shear strength

d^2 = depth of penetration into clay layer

mg = weight of cone

k_ω = cone Factor.

The proportional constant (k_ω) is primarily dependent on the angle of the cone and the sensitivity of the clay. The cone factor (k_{30}) for a cone of angle 30° was taken to be 0.85.

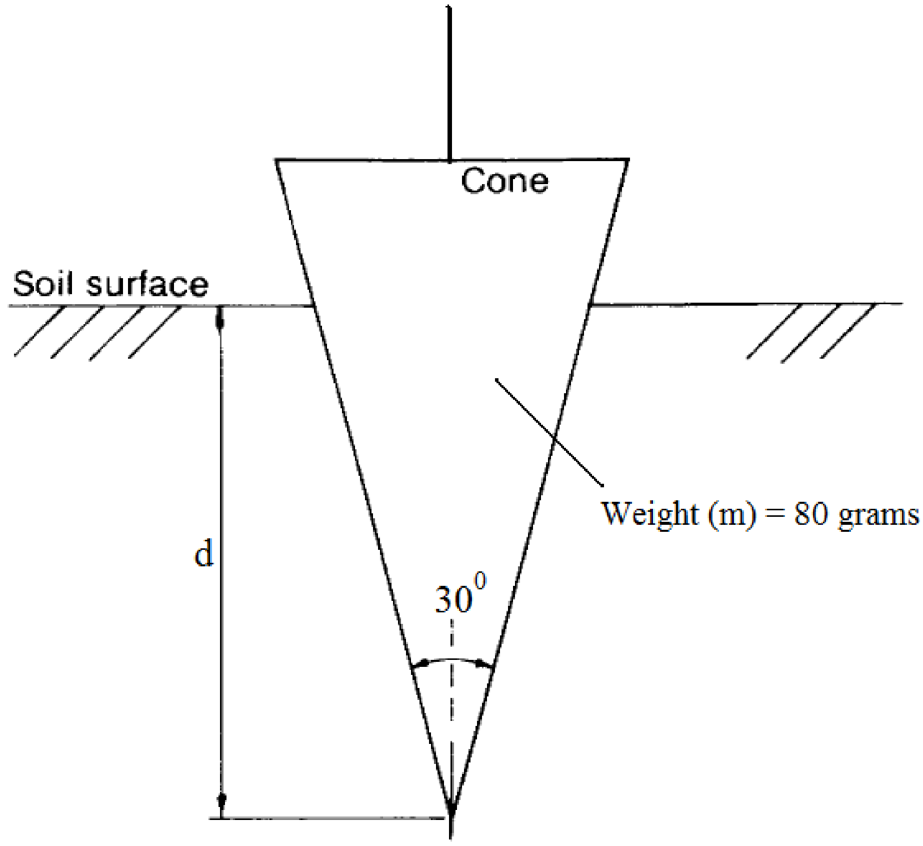


Figure 3.4: Fall Cone assembly (Adapted from BS 1337- Part 2: 1990)

The clay samples used for this test, were produced by hand mixing 0.5kg of dry powdered kaolin with distilled water until a thick homogenous paste was achieved (approximately 15-20 minutes of continuous mixing was necessary, depending on the intended moisture content).

After mixing, the clay was placed into a mould of 55mm diameter and 40mm depth and positioned under the cone; which was then released to begin testing. The test was performed 3 times on each of the samples, with the average penetration depth being used to calculate soil strength. Once the operation was complete, moisture content samples were extracted from the clay soil surrounding the area where the cone was dropped.

The tests were carried out for a range of clay moisture contents, with the oven drying method being adopted to determine moisture content. The liquid limit corresponded to the moisture content at which 20mm cone penetration was achieved (Muir Wood, 2009).

3.4 CLAY MIXING PROCESS

3.4.1 Initial Mix Testing

There were a number of methods which could have been adopted to obtain clay samples from the dry powdered kaolin, however due to time and equipment constraints on the project these were not undertaken. The methods deemed the most appropriate were hand mixing and machine mixing. These two methods were initially used to produce a range of samples to different moisture contents, in order to establish which method was best for achieving homogenous samples.

Hand mixing was selected as the most appropriate method because of the inability of the mixing machines to achieve homogenous samples below the liquid limit, i.e. large sections of dry powdered kaolin remained visible within all mixes when machine mixing was performed.

However, a consequence of adopting hand mixing to produce the clay was the inability to obtain homogenous samples below a moisture content of 40%; mixes consisted of dry powdered kaolin and lumps of moist clay, with no uniform texture. Another problem which became apparent was that for samples above 60% moisture content, too much water remained in the mix as pockets of free water were visible. The decision was therefore taken to investigate the performance of the dry cement inclusions in samples consisting of 40% and 60% moisture content respectively, as samples were workable during hand mixing and provided homogenous samples of consistent moisture results.

3.4.2 Adopted Mixing Process for Producing Clay Samples

Dry powdered kaolin was weighed to a known mass and placed into a large mixing bowl (Figure 3.5: Step 1). The quantity of water added was dependent on the intended moisture content, for example a clay soil (intended moisture of 40%) would require 400ml of water to be added for every 1kg of kaolin.

Once water was added to the dry powdered kaolin (Figure 3.5: Step 2), immediate mixing was performed with the use of a palate knife. This was done until all 'free' water was visually consumed by the powder.

Dry powdered kaolin (of known mass) placed into mixing bowl

Water introduced into mixing drum before mixing is undertaken with a palate knife

The kaolin and water are then mixed by hand in order to create homogenous clay material

Exploded view of kaolin – water composition during initial stages of mixing

After approximately 1 hour of continuous mixing, the result is homogenous clay.

Exploded view of moulded clay material produced from hand mixing

Figure 3.5: Hand mixing procedure

This operation was only required for the first minute or so after adding the water. After which time hands were used to knead the mixture (Figure 3.5: Step 3). Rubber gloves were worn during this kneading process, as this limits the possibility of water being evaporated by the temperature of the hands; thus altering the moisture content. The mixing process was performed for approximately 1 hour until all powdered kaolin was saturated by the water in the mix, thereby providing a homogenous sample of uniform moisture and consistency (as shown in Figure 3.5: Step 4).

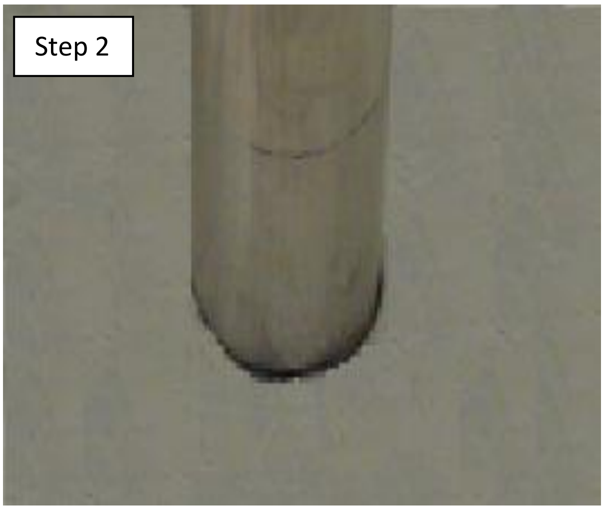
After the mixing operation was complete, the clay was placed into an appropriately sized perspex container. All clay samples mixed were in a plastic state so were easily moulded to the shape of the box and around the steel piece used to form the hole for the inclusion (see Figure 3.6: Steps 1 & 2). Once the clay surface level reached the required height, the steel piece was removed to form a cavity hole in the soil (Figure 3.6: Step 3).

Initially, dry cement was passed through a 600μm sieve in order to remove any large grains of unhydrated cement; which had gathered in the sealed bag during transportation of the cement to the University. The sieved cement powder, of known weight, was poured into the cavity until level with the top surface of the clay (Figure 3.6: Step 4). At this stage the sample surface was covered with cling film and the container lid sealed shut with the use of insulation tape (Figure 3.6: Step 5), to stop the possibility of water escaping from the surface through evaporation. The samples were then stored in a relative humidity room set to a humidity of 95% to 99% (Figure 3.6: Step 6). This was done in order to ensure that any change in moisture content was a direct result of water being drawn from the soil by the dry cement ensured as no evaporation, or soil drying, can take place in this non-drying environment. As a precaution, samples were weighed prior to and on removal from the humidity room to ensure that there was no moisture loss during curing.

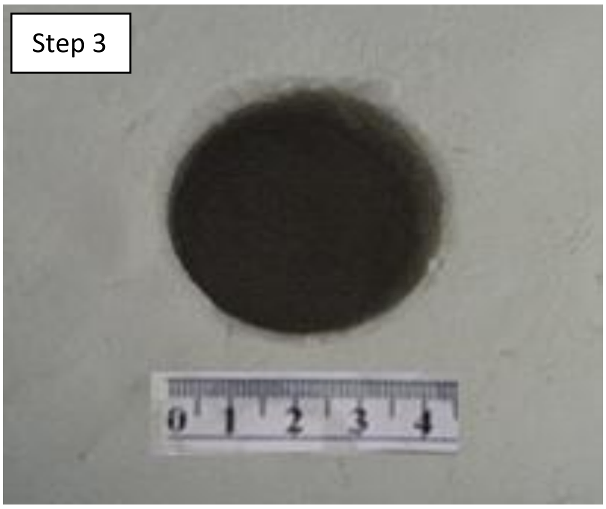
Samples were stored in the humidity room for 1, 3, 7, 14 and 28 days, which are common curing periods adopted for monitoring the strength gain of concrete samples.

For each mix, control samples were used to ensure that the clay was homogenous and at constant moisture content. The exact same conditions were placed on the control sample, with the only variable being the absence of an inclusion of dry cement.

Clay being compacted into perspex container on completion of hand mixing operation



Clay moulded around the perimeter of the steel piece



Steel piece removed to create cavity in the clay (Plan View)

Cavity is subsequently filled with loose dry cement (cement weighed prior to testing)

Sample is covered with cling film to ensure no moisture loss in the sample

Samples stored in a non-drying atmosphere (relative humidity ~ 95-99%)

Figure 3.6: Formation of cement column and storage conditions

3.5 CLAY TESTS

The moisture content is a frequently used mass ratio which simply depicts the mass of water (evaporated by heating to 105°C until constant weight); to the mass of dry soil (Muir Wood, 2009) where:

$$w = \frac{Mass\ of\ water}{Mass\ of\ dry\ soil}$$

The moisture content has a significant effect on the behaviour of the soil and is an important consideration when discussing soil strength and deformation characteristics.

3.5.1 Liquidity Index (LI) and Its Relation to Soil Shear Strength

The liquidity index is a useful parameter which describes the current state of the soil with respect to the limiting states found in the Atterberg test and can be defined as:

$$LI = \frac{w - PL}{PI}$$ **Where:** w = natural moisture content (%)

From the liquidity index equation, soils with a moisture content near or equal to the plastic limit can be determined to have a LI $\approx$ 0. These tend to be very stiff heavily overconsolidated soils (Ingles, 1987) with undrained shear strength of 170kPa. Soils with moisture nearer the liquid limit will have a LI $\approx$ 1, and tend to be very soft normally consolidated soils of undrained shear strength equal to 1.7kPa (Muir Wood, 2009). Using this, a relationship between the liquidity index and the logarithm of undrained shear strength can be derived (as shown in Figure 3.7).

The above theory was applied within this study to relate the liquidity index of clay samples to some corresponding shear strength. However, as this relationship is itself not a direct measurement of the kaolin clay being used in this investigation, the relationship was verified by another means. Two methods selected to measure the shear strength of clay at different moisture contents were the standard fall cone test and hand shear vane tests (discussed further in Chapter 4).

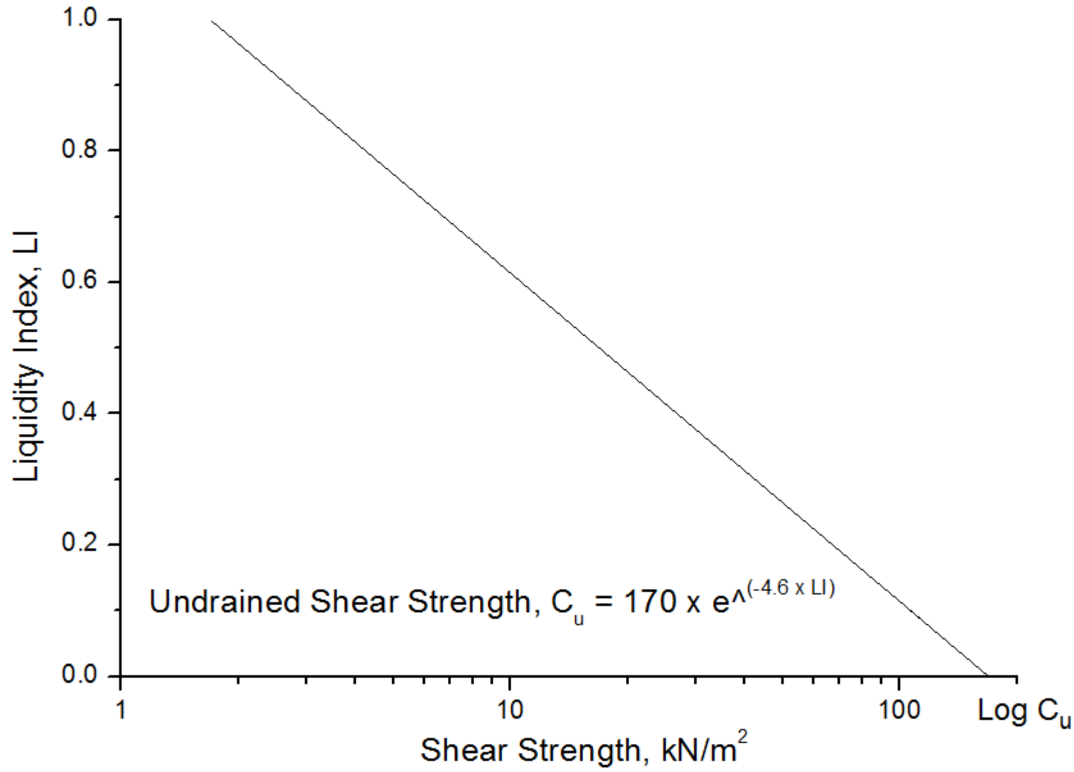
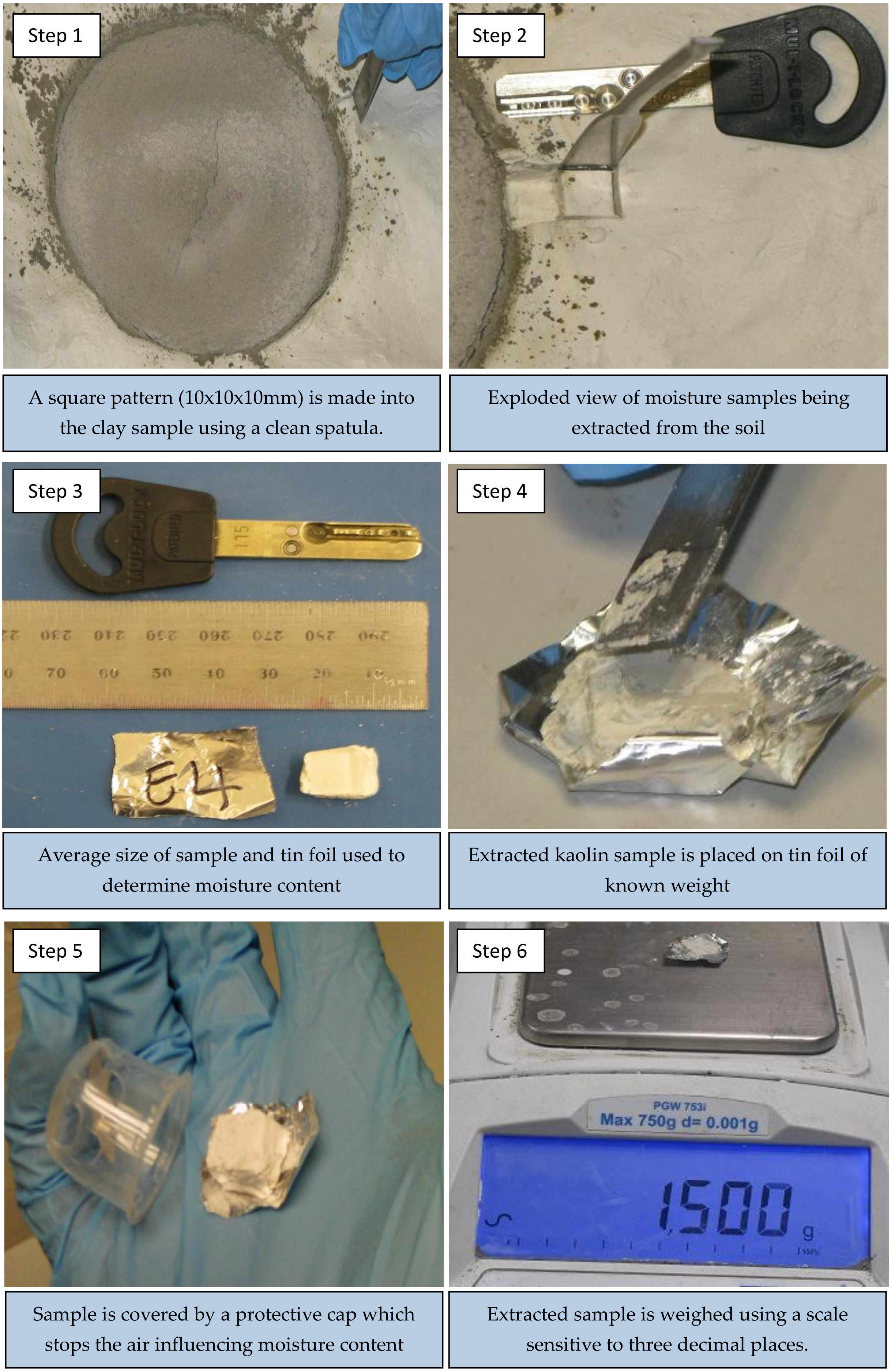


Figure 3.7: Shear strength and liquidity index relationship for clay (Atkinson, 2007)

Size of Moisture Content Samples

In order to determine the radial dewatering effect experienced by the soil as a result of the dry inclusion utilising pore water in order to hydrate, moisture samples were extracted from the inclusion-soil interface at increasing radial distances and at increasing depths from the soil surface (Figure 3.8: Steps 1 & 2). The radial dewatering of the soil was then determined by examining the difference in moisture content obtained for samples containing an inclusion, in comparison to a control sample.

The samples extracted for moisture content testing were 10 x 10 x 10mm in size (Figure 3.8: Step 3), with the moisture content assumed to be at the central position of the sample. Taking samples this size provided weights in the region of 2 to 3 grams depending on the volume of moisture in the sample. This weight is less than the 10 grams recommended in BS 1377. However, the sample sizes were considered appropriate as they provided consistent moisture content results when extracted from control samples. The variation in control sample results ranged from control moisture (%) ± 2%, with these variations encountered at few locations in the sample.



Step 1: A square pattern (10x10x10mm) is made into the clay sample using a clean spatula.

Step 2: Exploded view of moisture samples being extracted from the soil

Step 3: Average size of sample and tin foil used to determine moisture content

Step 4: Extracted kaolin sample is placed on tin foil of known weight

Step 5: Sample is covered by a protective cap which stops the air influencing moisture content

Step 6: Extracted sample is weighed using a scale sensitive to three decimal places.

Figure 3.8: Moisture content samples extracted from kaolin

Extracting samples smaller than 10 x 10 x 10mm caused a wide variation in moisture content value; with control samples of 40% moisture content displaying results of 40% ± 10% in some cases. This was not acceptable as the control sample should display consistent results if homogenously mixed.

In order to accurately calculate the moisture content, it was important to have as much sample as possible relative to any other weight measurements. Such measurements include the weight of the equipment used to hold the sample; as this is taken into account in the equation for moisture content. Tin foil was deemed a suitable material for holding the moisture samples about to be weighed, as it is extremely light and does not influence the moisture content calculation (tin foil shown in Figure 3.8: Step 4). The tin foil adopted for this study weighed an average of 0.025 grams: which contributes to an averaged 1.04% of the weight, with respect to the moisture samples.

Immediately after extraction, the samples were weighed in order to obtain the wet mass – mass of solids plus mass of water. Care was taken to ensure that the sample did not have time to dry in the atmosphere by covering the sample with a protective cap (Figure 3.8: Step 5); as drying would have altered the moisture content. Once weighed the sample was placed into an oven set to 105^{o}C, until constant weight over a 24 hour period was achieved. The sample was weighed immediately after removing from the oven (Figure 3.8: Step 6).

Once all necessary moisture samples had been extracted and weighed, shear vane tests were performed at increasing radial positions from the inclusion-soil interface. These tests were carried out in undisturbed areas of the clay soil (as discussed later).

3.5.2 Shear Vane Test

Shear vane tests were carried out in accordance with BS 1377-Part 7: 1990. For the purposes of this investigation two different forms of vane apparatus were used. For samples prepared to moisture content of 60% a motorised vane (Figure 3.9) was the preferred choice, as this was sensitive to the low shear strengths experienced at this moisture. For samples prepared to moisture content of 40% a hand vane was used, due to the shear strengths of these samples exceeding the capacity of the motorised vane.

The hand vane is not as sensitive as the motorised vane as the speed of rotation cannot be applied at a controlled rate. In order to obtain some control over this factor the shear vane was operated solely by the Author of this report. However, as this test is intended as a quick confirmation of strength improvement, the inability to obtain a constant rotation rate was not a major concern.

Another variation between the two vane systems was the difference in vane dimensions. BS 1337-Part 7: 1990 provides strict conditions for the vane to meet in order to be considered appropriate for use. The dimensions of the vane are the most important design aspect and must conform to the Area Ratio, which ensures as little remoulding and disturbance is caused to the soil as possible.

$$Area\ Ratio = \frac{8T(D-d)+\pi d^2}{\pi D^2} x\ 100$$

Where: D = overall blade width measured to 0.1mm, T = thickness of the blades measured to 0.01mm, d = diameter of the vane rod, including any enlargements due to soldering measured to 0.01mm.

The Area Ratio should not exceed 15%. The mechanical vane used in this investigation has an Area Ratio of 6%, with 10% calculated for the hand vane; therefore both vanes were appropriate for use.

The vane was penetrated to a depth half the inclusion length and to different radial distances from the inclusion (as shown in Figure 3.10). This allowed a quick indication of the soil shear strength to be determined at these points.

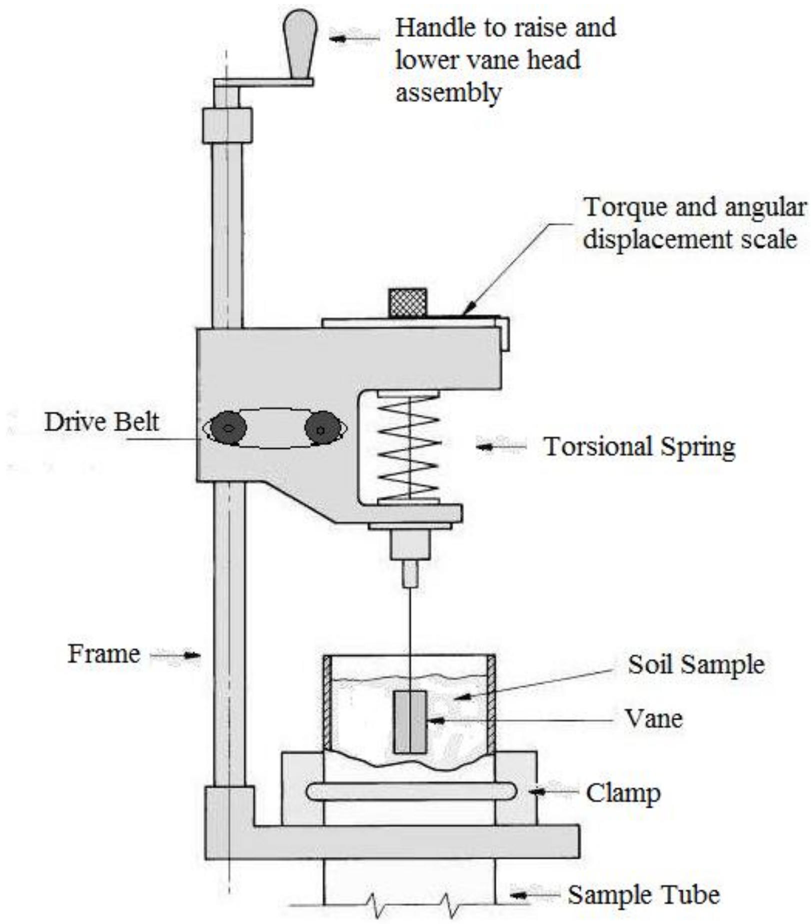


Figure 3.9: Mechanical shear vane (Adapted from BS 1377-7: 1990)

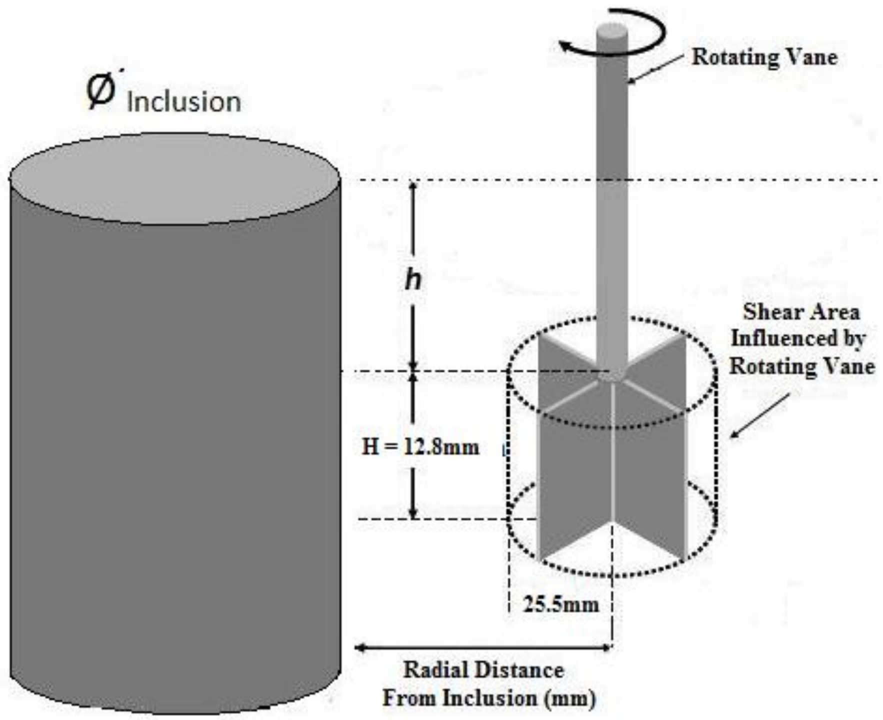


Figure 3.10: Mechanical shear vane operating near hardened inclusion

3.5.3 Filter Paper Suction Test

The intention for the dry cement inclusions proposed in this research is to induce a dewatering effect within the surrounding soil, as a result of the cement utilising the soils pore water in order to hydrate. It has been well established that a reduction in the soils moisture causes suction forces in the soil to increase, as a direct result of capillary forces acting in the soil matrix (Leong, 2008). It is also known that suction can affect the mechanical behaviour of the surrounding soil; increased suction causes the draining of water out of the soils pores to become more and more difficult (Haut, 2005). It is hoped that the change in the soils mechanical behaviour as a result of a reduction in the soils pore water can be investigated through the use of suction tests. There are many methods to determine suction forces, however due to its ease, cost and relatively simple practices the filter paper suction test has been selected for this investigation.

The filter paper suction test is an 'indirect method' of measuring the two components which produce soil suction: total and matrix suctions. Matrix suction is recognised as representing the energy (j) of soil water per unit volume (m^3) and its measurements are based on a close contact between the filter paper and soil (Lu, 2008). The measurement is based on the moisture content of the filter paper representing the magnitude of matrix suction.

For the measurement of total suction, filter papers were simply placed on top of a PVC O-ring, which ensured no contact between the filter paper and clay specimen was possible. This leads to the filter paper acting like a semi-permeable membrane which is only permeable to water vapour. As no contact is made with the soil, the transfer of ions (in the pore water) through the filter paper is not possible.

The test is based on the assumption that an initially dry filter paper will equilibrate, in terms of water or vapour flow for soils with a specific suction. That is pore fluid (water or vapour) will transfer from the soil to the filter paper, and will continue to flow until suction in the filter paper and soil reach equilibrium. On establishing equilibrium the filter papers are removed and the moisture content of the filter paper are determined as quickly as possible, using scale balance sensitive to 0.0001gram accuracy. This is due to the small increases in moisture observed. Using the values of filter paper moisture content and the appropriate calibration curve, the soil suction can be determined (Bulut, 2001). For the purposes of this investigation the wetting curve provided by (Rahardjo, 1993) was used to determine suction (Figure 3.11)

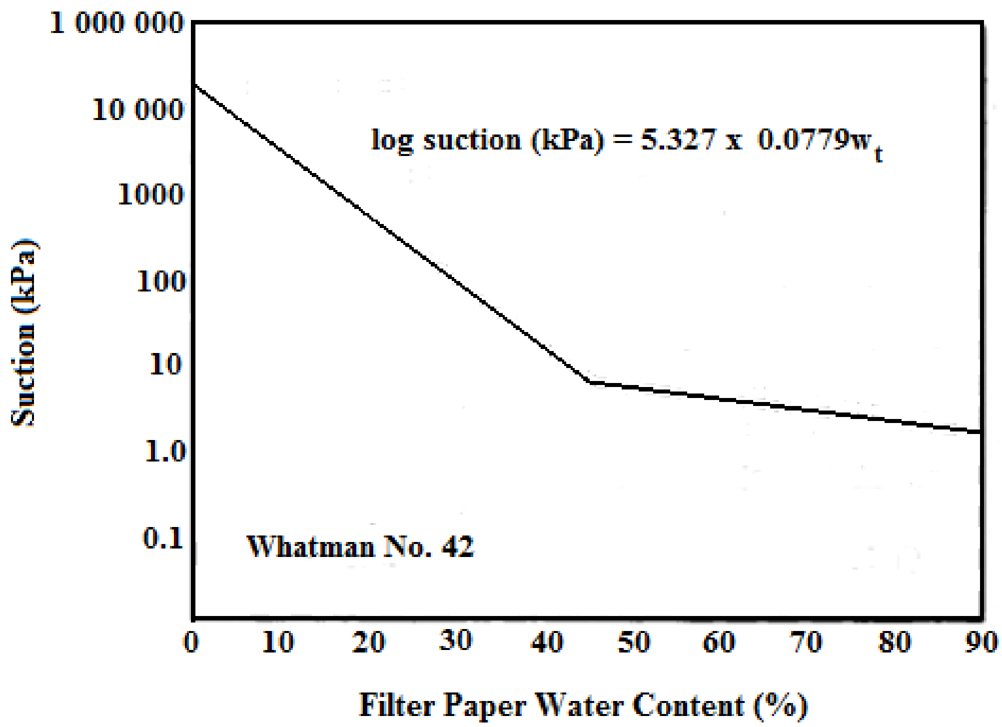
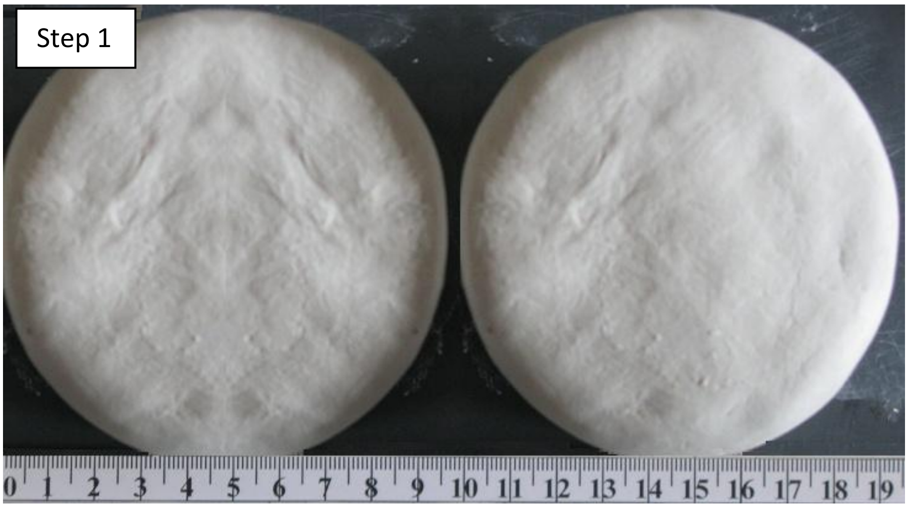


Figure 3.11: Calibration curve for filter paper suction test (Rahardjo, 1993)

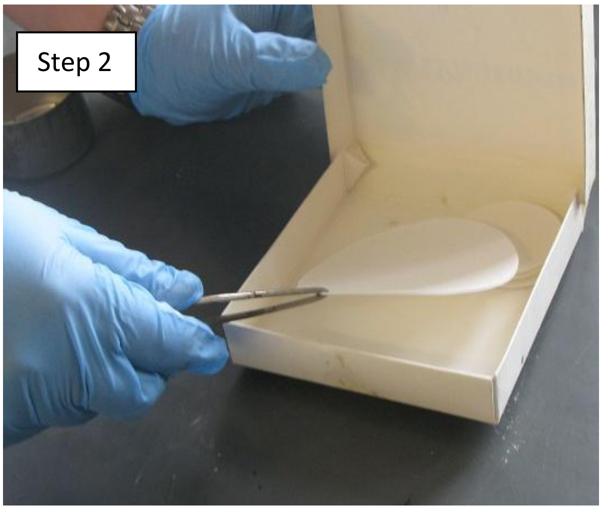
Two different situations will be analysed during these tests to the determine soil suction. The first test specimen will act as a control sample, with matrix and total suction measurements being determined for a clay sample (as mixed in section 3.4.2) with water content equal to 40%. As described matrix suction will be determined through equilibrium of filter paper in close contact with the soil, with total suction measurements recorded at some elevation above the top surface (Figure 3.13).

The second situation will focus on the changes in suction experienced as a result of introducing an inclusion of dry cement. Suction values at different locations from the cement-soil interface will be studied in order to enhance our understanding of the cements involvement in water flow through a clay sample (Figure 3.14). It was anticipated that the dry cement would have a large influence on the suction values recorded throughout the clay sample, as the cements natural affinity for water should lead to a dewatering effect and changes to suction throughout the sample.

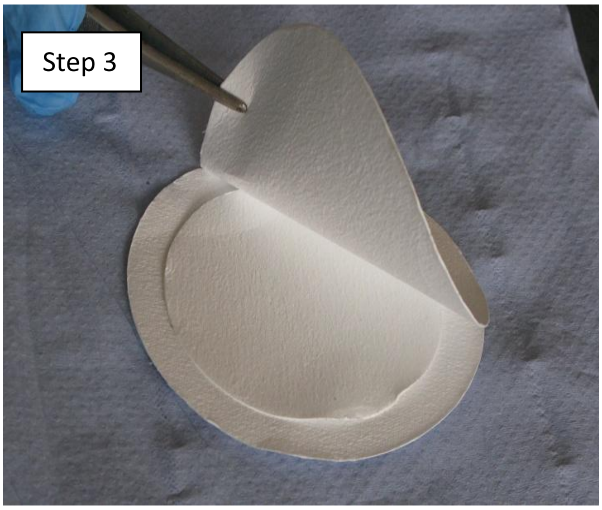
The tests were performed as detailed in Figure 3.12, with the containers stored in a sealed humidity room as this ensured no temperature fluctuations were encountered. The samples were left to equilibrate in the humidity room for 7 days; which is widely deemed as an acceptable period to achieve equilibrium (Rahardjo, 1993).



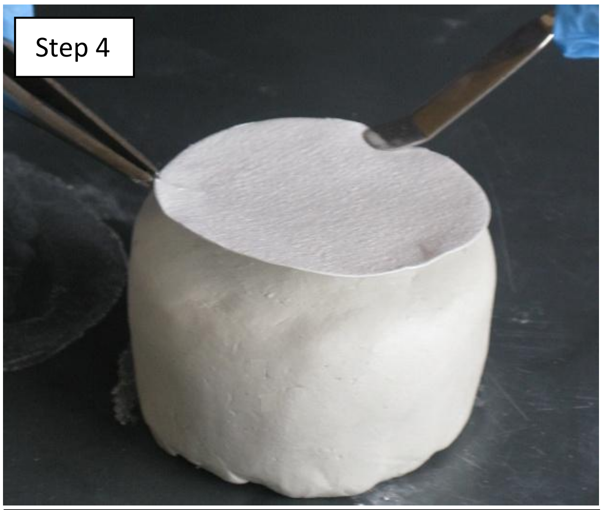
100mm sample was extracted from kaolin sample (as mixed in 3.4).
The sample was cut into two halves for matrix suction tests.



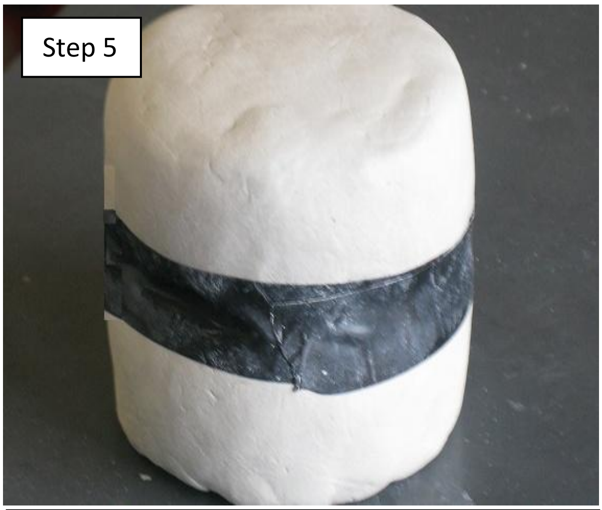
Removed Schleicher & Schuell No. 589-WH filter paper from box using tweezers



Placed filter paper between two protective filter papers of larger diameter



The sandwiched filter paper was placed on top of one half of the clay samples using tweezers



Second sample piece was then placed on top and taped to bottom piece with electrical tape

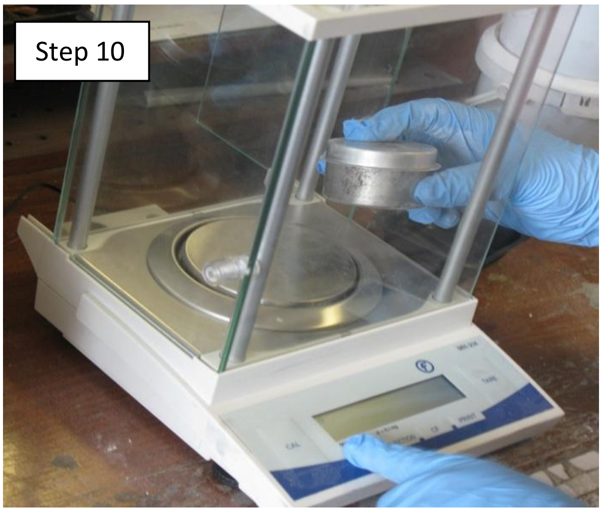
Figure 3.12: Filter paper suction test methodology

Sample placed into container, with a clean PVC O-ring inserted for total suction

Two Schleicher & Schuell No. 589-WH filter papers were then placed on top of the O-ring.

Containers were wrapped and placed into 95-99% RH room with constant temperature.

Immediately after opening jar, the filter paper on the O-ring was removed using tweezers.



The wet filter paper was placed into moisture tin and weighed for total suction.

Using a scale to 0.0001g accuracy the dry, cold weight of the moisture tins was taken.

Figure 3.12: contd………………

Filter paper sandwiched between protective papers was again removed using tweezers.

Filter paper was immediately placed into moisture tin and sealed shut with tin lid.

Tin with wet filter paper inside was weighed for matrix suction measurement.

All tins placed in oven (105^{o}C), with lids half open.

After removing from oven a large aluminium block is used enhance the cooling process.

The tin is again weighed, this time to determine dry weight of filter paper.

Figure 3.12: contd..................

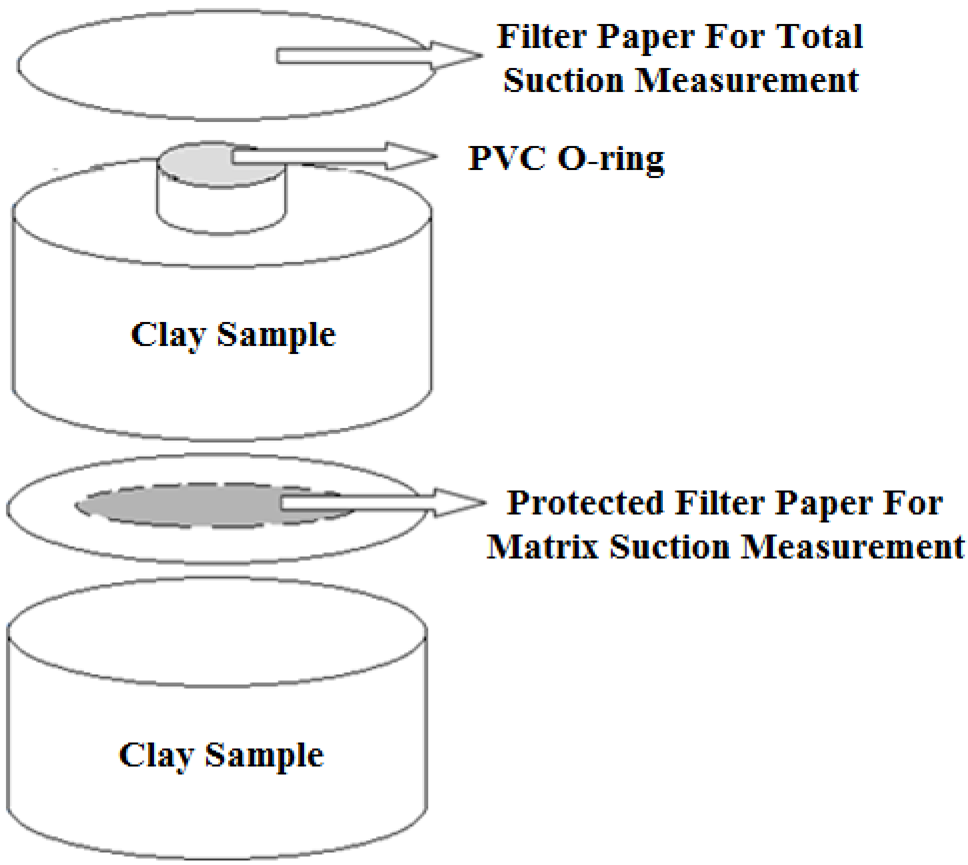


Figure 3.13: Traditional filter paper suction test sample setup

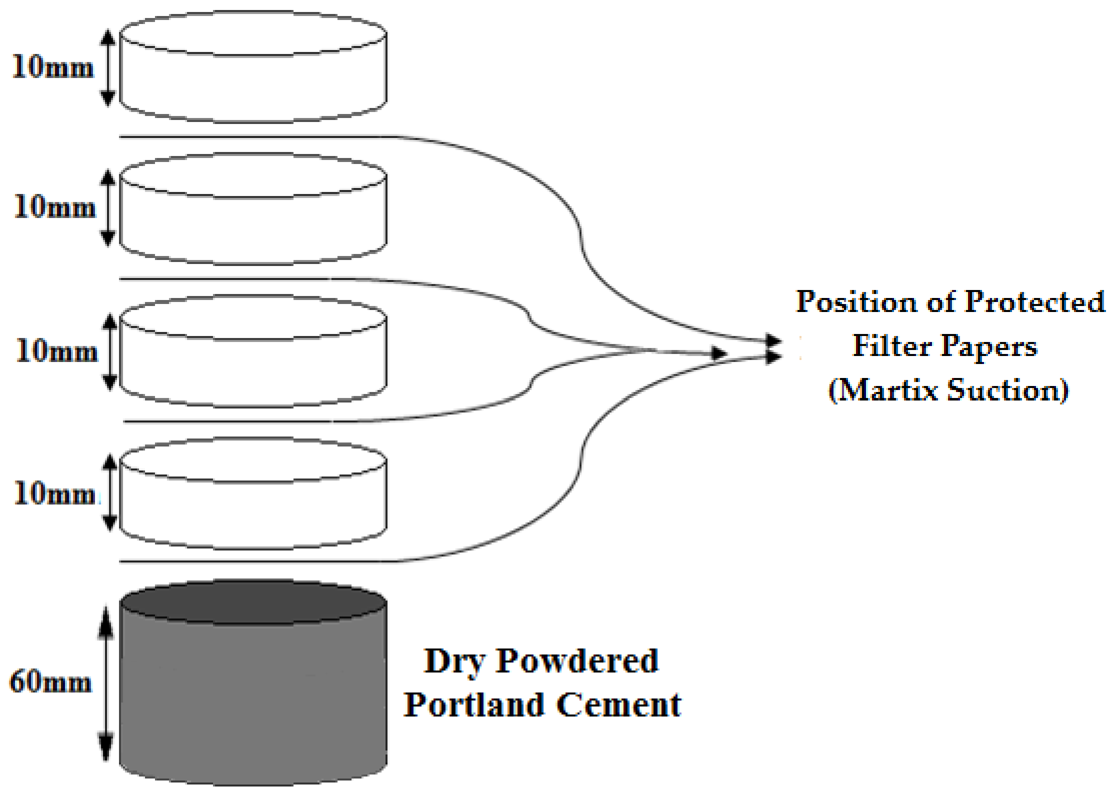


Figure 3.14: Authors adaption to filter paper suction setup

3.6 CEMENT INCLUSIONS IN CONSOLIDATED CLAY

In situ clay soils by their very nature have been subject to different forms of stress throughout their history, due to erosion and deposition, with the stress history having a significant influence on the soils behaviour.

An attempt to replicate stress conditions similar to those experienced by *in situ* clay will be made. This will involve consolidating and swelling samples under known stress regimes to create normally consolidated and overconsolidated samples. Again the dewatering effect experienced by clay in the presence of dry cement will be assessed.

3.6.1 Initial Consolidation Tests

Tests were carried out to determine the best practice for consolidating kaolin clay samples. Several samples were initially mixed to a moisture content equal to 120% and placed into a consolidation cell. Each sample was left in the rig for a period of 5 days, with different consolidation pressures of 100kPa to 200kPa being applied across the samples. The samples could have been consolidated for a longer period, however 5 days was intentionally selected due to time restrictions placed on the investigation.

From these tests, it was clear that all samples consolidated below 140kPa were not suitable for testing as the soil was extremely soft and easily disturbed when handled. The samples pressured to 140kPa were more appropriate with consistent moisture content values recorded throughout the sample. Pressures above 140kPa also provided consistent results however with reduced moisture contents. As samples consolidated to 140kPa were satisfactory, in terms of its handle ability and consistency, coupled with average moisture contents of 54% recorded throughout the sample, this pressure was arbitrarily selected for testing.

3.6.2 Sample Preparation and Execution

Dry kaolin of known mass was weighed and placed into a mixing bowl (Figure 3.15: Step 2). Distilled water was added to achieve a moisture content of 120% (well in excess of the liquid limit), with initial mixing being carried out with the aid of a wooden spoon (Figure 3.15: Steps 3 & 4). The sample was mixed until all dry powdered kaolin had been visually removed. At this stage the mix was transferred to a mixing machine, which provided continual mixing of the kaolin for a period of approximately 1 hour or at least until a homogenous mixture was achieved (Figure 3.15: Steps 5 & 6).

The mixture was then carefully transferred into a consolidation cell, which was attached to a pore pressure transducer at the base. During this process it is important to maintain a constant flow of the liquid kaolin into the cell, as inconsistent or careless transfer can lead to voids forming throughout the sample. Tipping the cell to some angle, when pouring the liquid kaolin, was adopted to assist consistent and continual flow (Figure 3.15: Step 8).

Once all the mixture had been transferred from the mixing bowl, the cell was placed under a piston; which was connected to an actuator (Figure 3.15: Steps 9 & 10). As can be observed in Figure 3.15: Step 10, the piston was fitted with a porous disk to allow drainage. Through increasing the inlet pressure to the actuator, the piston was lowered onto the top of the kaolin sample, causing the stress to be transferred from the piston to the sample. Water was initially prevented from escaping the cell, which caused the pore pressure to increase in order to counteract the load from the piston. The piston was lowered until the pore pressure transducer read the desired pressure of 140kPa (Figure 3.15: Step 12). At this stage, the water was permitted to leave the cell by opening drainage taps at the top and bottom of the cell (2 way drainage). This allowed the water to drain and the load from the piston to be transferred to the kaolin particles. The sample particles compressed and reduced in voids ratio as a result of this load transfer.

Leaving the samples to consolidate for a period of 5 days, the load was then removed from the sample, by removing the inlet pressure to the actuator. The sample was then transferred into a specially designed rig (detailed later) with different loading conditions being reapplied to the sample.

Control samples were created to ensure the consolidated kaolin produced a consistent moisture content profile along its length, with moisture content again being determined through the oven dry method.

Powdered kaolin before testing

Powdered kaolin is placed into a standard mixing bowl

Water is added, volume depending on volume of dry kaolin, to achieve a moisture = 120%

Hand mixing is performed for period of 5-10 minutes until all dry kaolin is saturated

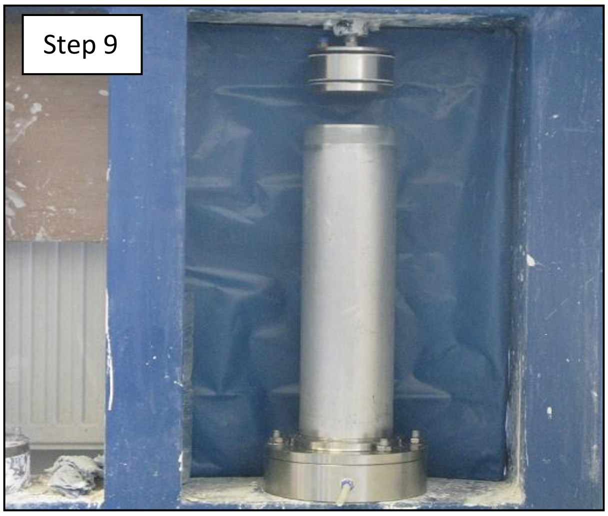
After hand mixing, kaolin is transferred to standard mixing equipment until homogenous

Homogenous consistency of kaolin at 120% before being placed in consolidation rig

Figure 3.15: Process adopted for consolidating samples

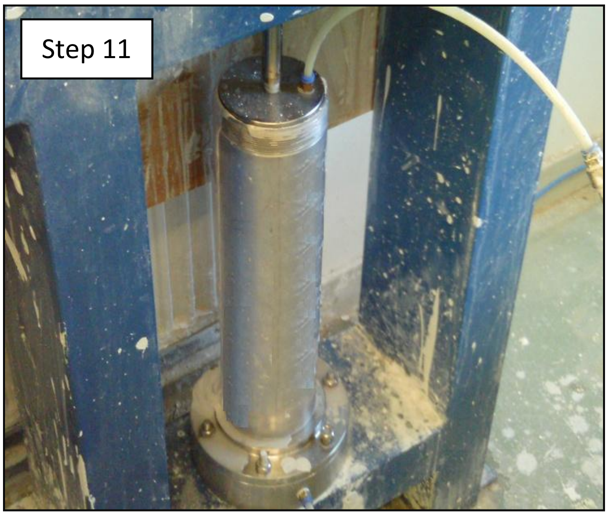
Permeable base which is connected to a pore pressure transducer

Liquid kaolin is poured into consolidation cell with care to avoid formation of air voids



Consolidation cell positioned below piston

Porous disk at the base of the piston



Piston is lowered onto top of sample with drainage denied to increases pore pressure.

Pore pressure is monitored with pore pressure transducer.

Figure 3.15: contd..........

3.6.3 Rig Design

As mentioned, the stress history of clay can have a significant influence on its behaviour. To observe the behaviour of cement in contact with normal and overconsolidated clay soils; on the ability of dry cement to utilise pore water from the clay, the design and manufacture of a rig (Figure 3.16) was commissioned by (Brown, 2010).

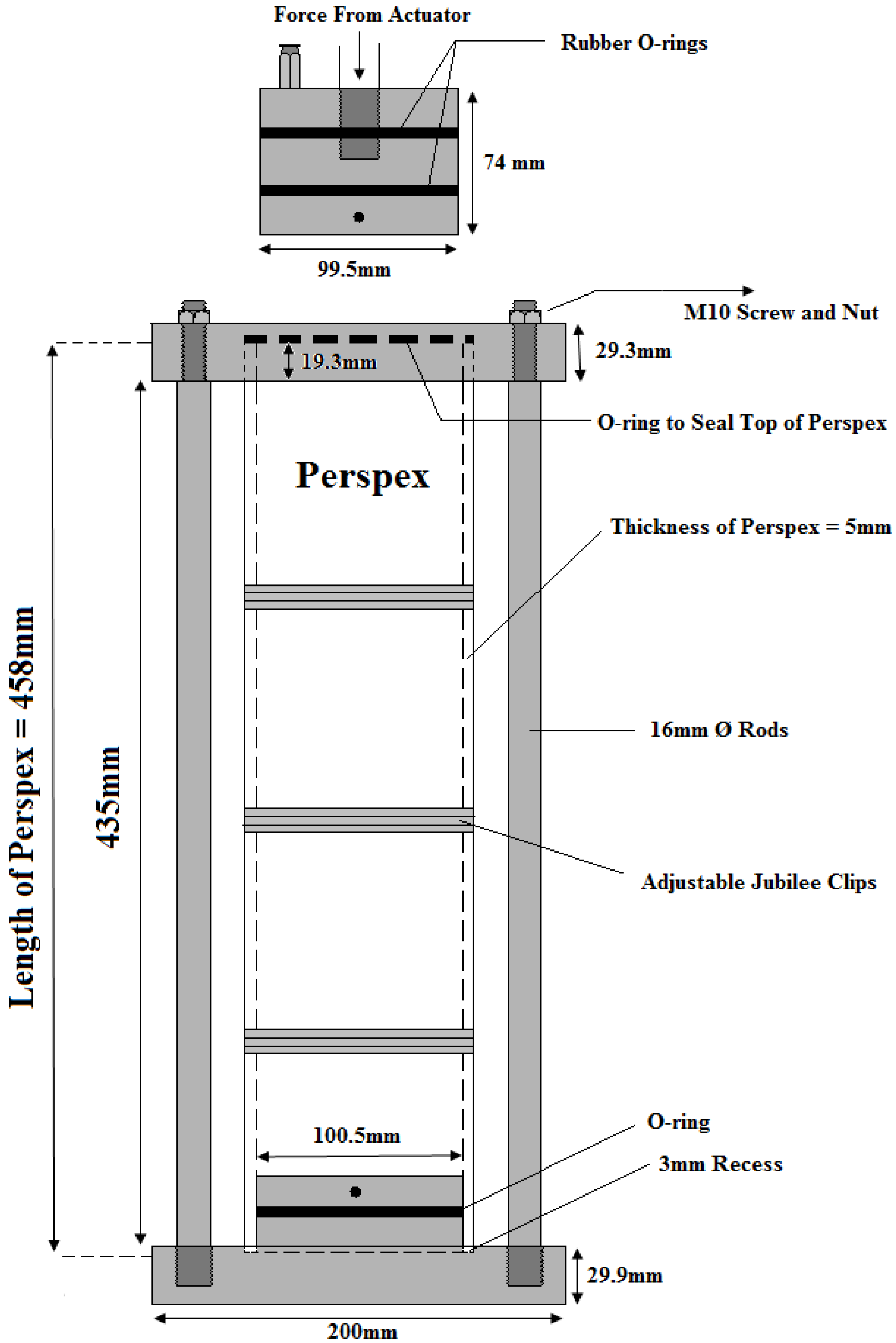


Figure 3.16: Schematic of rig components

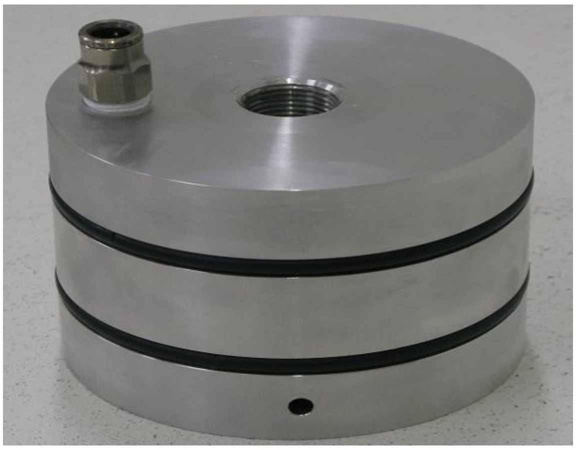


Figure 3.17: Solid impermeable aluminium piston

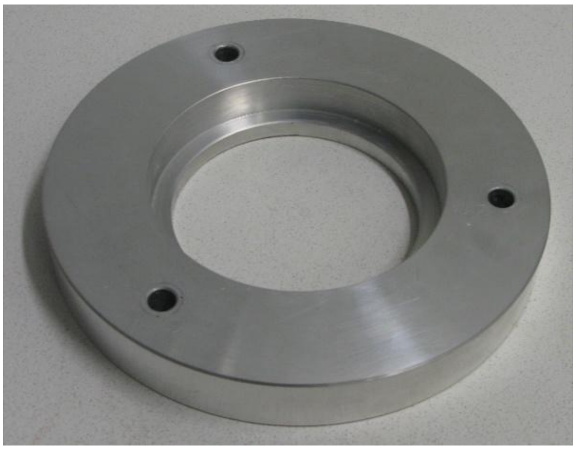


Figure 3.18: Underside of top section of rig

Figure 3.19: Solid impermeable aluminium base

Figure 3.20: Full assembly of rig components

The main purpose behind this rig was to keep the clay sample under known stress and allow the interaction between the soils pore water and the cement to be investigated in a controlled manner.

Design Aspects and Equipment Used to Form the Compression Rig

The Author was seeking both a visual inspection and physical data from the tests performed in this rig, as this would provide a better understanding of the mechanical interactions that exist between the dry cement inclusion and the pore water in the clay soil. Design aspects similar to the consolidation rig (Figure 3.15) were incorporated as the samples would initially be loaded using this equipment.

Focusing on the visual aspect of the proposed tests the decision was taken to use a clear perspex tube in the rig, instead of the aluminium U100 sample tubes used to consolidate the samples. The perspex was reinforced with three adjustable jubilee clips (Figure 3.16), which ensured that hoop stresses likely to be generated during testing would not cause the tube to bulge.

As the consolidated clay samples were being transferred between rigs, it was essential that no disturbance to the sample was experienced. In order to avoid this, the internal diameter of the perspex tube remained the same as the U100. Transfer of the clay soil to the perspex tube was possible using a hand operated jack, which pushed the clay sample from the U100 through to the perspex tube with minimal disturbance to the sample.

The base (Figure 3.21) of the rig was designed with a 3mm recess to provide a sealed fit with the perspex tube, with a single O-ring incorporated on the base to enhance the seal.

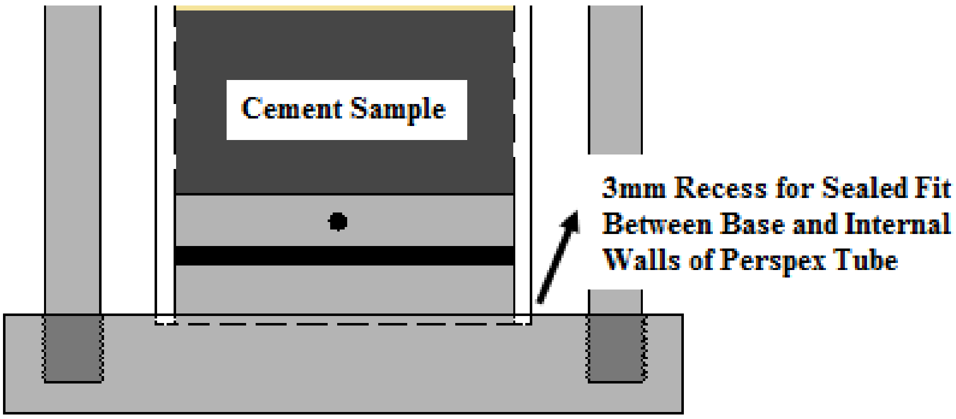


Figure 3.21: Recess in the base for sealed fit with internal perspex tube walls

The porous stone disc evident on the consolidation rig base (Figure 3.15: Step 7) was intentionally removed as this has the tendency to absorb and retain water. As dry cement would be in direct contact with the base of the rig, the concern was that dry cement at the base would hydrate as a result of water in the porous stone, instead of hydrating solely by the pore water from the soil.

The main purpose of this rig was to essentially monitor water movement from a clay soil into a cement layer and examine whether or not the cement had the ability to utilise pore water from soils currently experiencing lower stresses; in relation to a preconsolidation stress of 140kPa. To apply pressure to the sample a solid non-permeable aluminium piston was manufactured. The piston was designed to a diameter of 99.5mm, which facilitated the movement of the piston within the perspex tube, with two rubber O-rings incorporated to provide a sealed fit between the piston and internal walls of the tube (Figure 3.16).

A potential problem with the design of the piston involved the release of air from the tube during loading and unloading (Figure 3.22). Compressive forces would resist the movement of the piston during loading, whilst the vacuum would make the release of the sample more difficult during extraction.

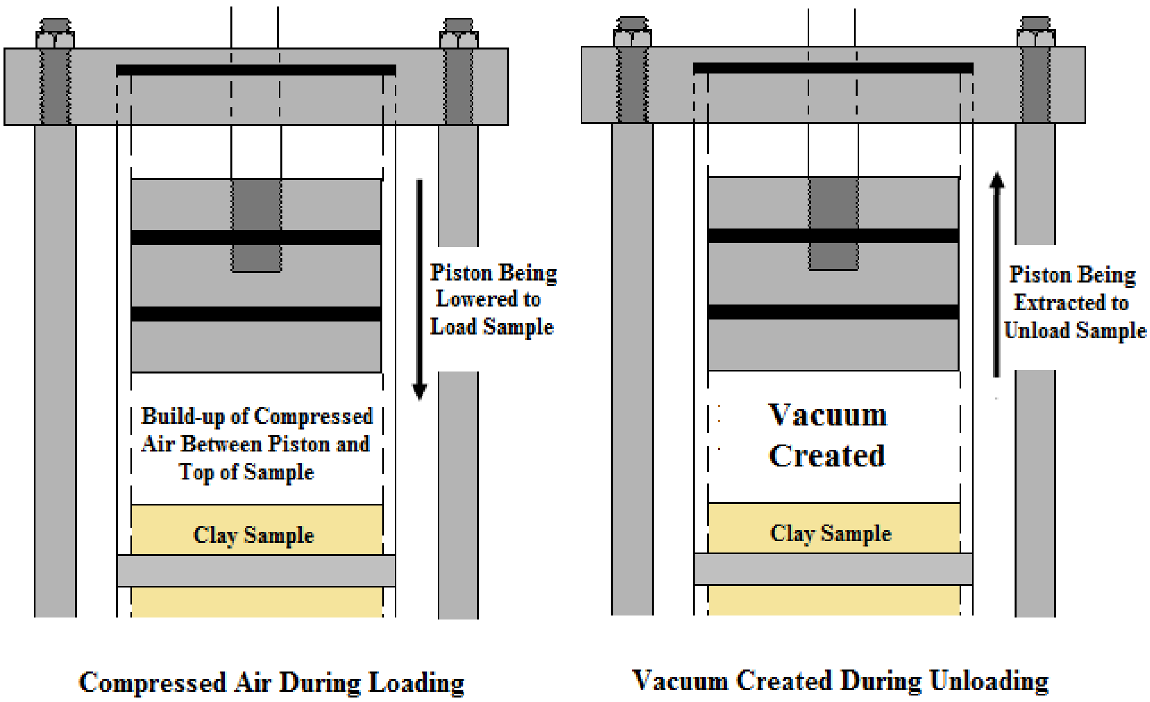


Figure 3.22: Potential problems with impermeable aluminium piston

In order to combat these problems, air release holes were manufactured into the piston design which facilitated the removal of air during loading and unloading (Figure 3.23).

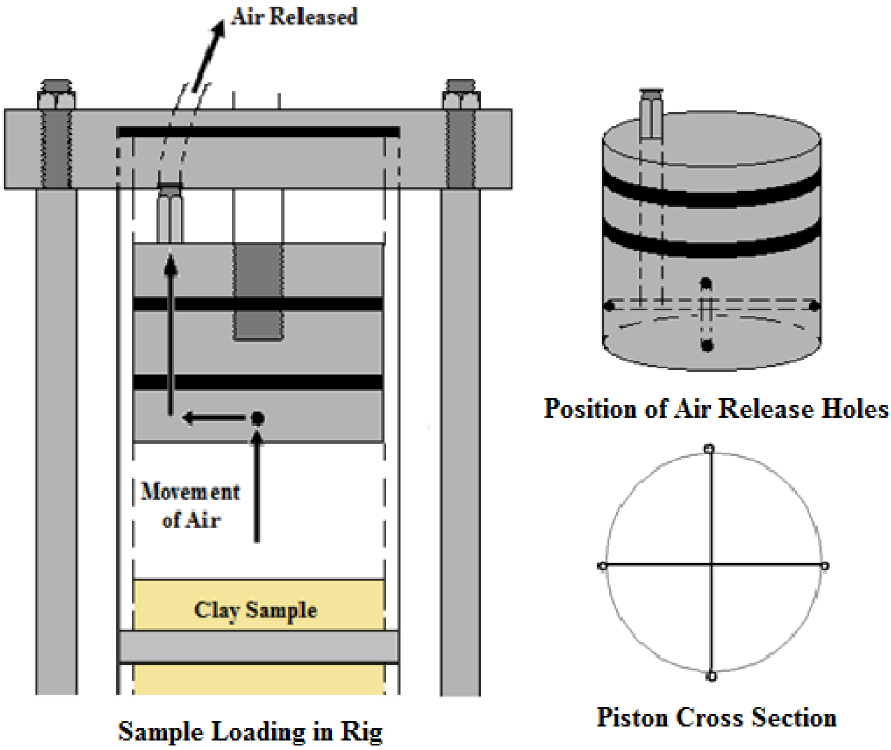


Figure 3.23: Air release holes to remove compressive/vacuum forces during sample loading/unloading

These air release holes were positioned below the rubber O-ring, as no air can pass through the seal between the O-ring and internal tube walls. Therefore, positioning these anywhere above the O-ring would have no benefit to the release of air from the tube.

After consolidating the clay for 5 days, the samples were transferred into the perspex tube with the aid of a hydraulic jack and placed to an arbitrarily marked location 60mm above the position of the top section of the base (as detailed in Figure 3.24). This was done in order to ensure that a cement layer 60mm thick could be consistently achieved for all tests performed in the rig. The clay samples were loaded and remained in constant contact with the dry cement for the appropriate duration, after which numerous tests (Table 3.8) were performed.

A visual inspection of the penetration of water through the dry cement with respect to time (indicated by progressive darkening of the cement), was observed for each test performed.

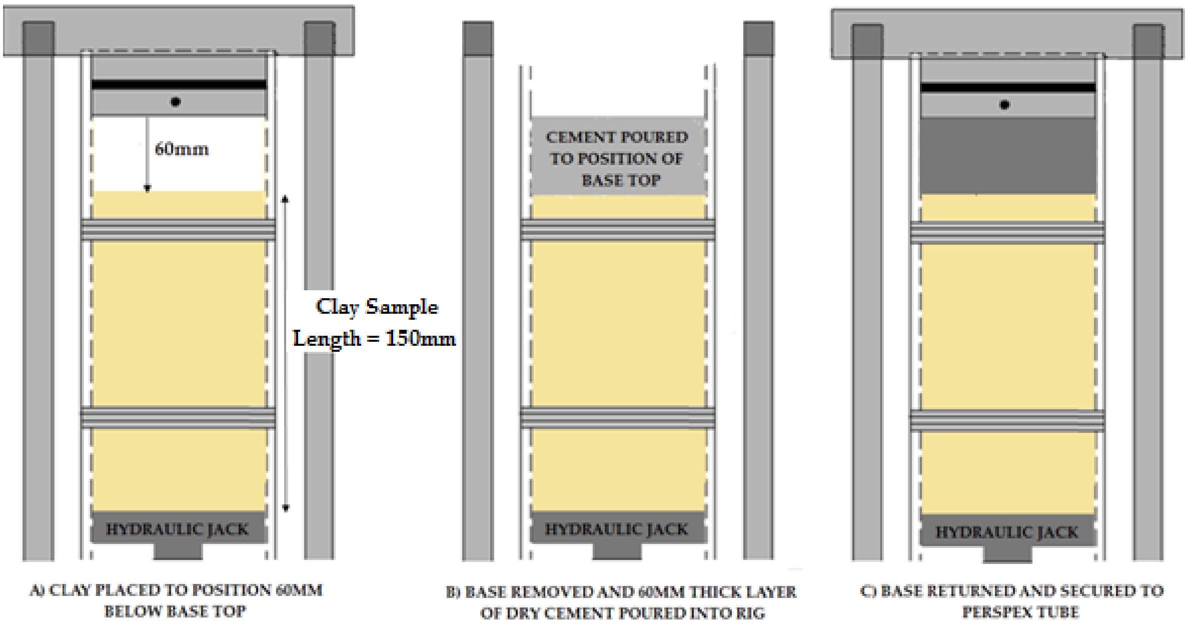


Figure 3.24: Method adopted to get cement layer into rig

Table 3.8: Various tests performed on clay and cement samples

Material Tested	Test Performed	Test Time, Days
Normally Consolidated Clay	Moisture Content Hand Shear Vane Moisture Content	1, 3, 7, 14 & 28
Cement in contact with a Normally Consolidated Clay	TGA[#] MIP[#] Nitrogen Adsorption[#]	
	Capillary Sorption[#]	
Overly Consolidated Clay (OCR[+] = 1.5)	Moisture Content	7
Overly Consolidated Clay (OCR[+] = 2.0)		
Cement in contact with an Overly Consolidated Clay	TGA	

[+]OCR – Over Consolidation Ratio

[#] Tests described in Section 3.7.1 to 3.7.5 of this report

The compression rig was calibrated using a standard load cell which was positioned between the ram of the pneumatic actuator and the loading piston in the centre of the rig. By continually increasing the pressure from the actuator to the piston the total stress which would be applied to the clay sample could be determined, by calibrating the pneumatic pressure readings against the load cell. An advantage to calibrating the rig in this manner was the presence of the frictional losses which occur during loading as a result of the O-ring and perspex walls.

The moisture changes experienced in the clay were tested by extracting moisture contents along the full 150mm length of the samples, i.e. at increasing radial distances from the cement-soil interface. Hand shear vane tests were also performed at different depths through the clay sample, in order to generate a profile of soil strength with respect to distance from the cement.

After performing the moisture and shear vane tests in the clay soil, and removing all kaolin clay from inside the rig, the cement samples were removed from the perspex tube by simply pushing the sample through the tube, aided by a hydraulic jack, until it was possible to lift the sample free.

Different OCR values were achieved by the loading the sample to stresses less than the preconsolidation stress and were investigated due to the likelihood of stresses relaxing in the soil surrounding when a hole is created to facilitate the inclusion. Arbitrary OCR's values of 1.5 and 2.0 have been selected for testing and were performed at 7 days.

3.7 CEMENT INCLUSION TESTS

3.7.1 Thermogravimetric Analysis (TGA)

In order to dispel the Authors initial concerns; regarding the inclusion skin effectively 'sealing off' the system to continual water penetration, Thermogravimetric Analysis (TGA) was used to evaluate whether or not the cement at the central core of the inclusion had access to water. By extracting cement samples at 5mm intervals from the interface of the inclusion to the central core, a profile of water penetration through the dry cement 'inclusion' (by monitoring hydration reactions) was generated.

TGA can be used to determine the degree of hydration by monitoring the weight loss of samples in a nitrogen environment with an increasing temperature from 20-1000^{o}C, with a temperature increase of 10^{o}C/min. This weight loss is associated with the transformation of $Ca(OH)_2$ to CaO at a temperature between 400 to 500^{o}C.

Control samples of w/c ratio equal to 0.5 were cast and cured in water at 20^{o}C until ready for testing at 1, 3, 7, 14 and 28 days. Thereafter, they were oven dried at 105^{o}C to effectively suspend hydration reactions within the paste, until constant weight was achieved over a 24 hour period. Samples installed in the kaolin clay were tested after the same number of days as the control samples. Prior to testing samples were ground to a fine powder using a ball mill, from which 50mg samples were obtained.

3.7.2 Paste Setting

Paste setting time tests were carried out in compliance with BS EN 196-3, with five specimens of 100% Portland cement, mixed to a water/cement (w/c) ratio of 0.5, being tested. An average time for the pastes to harden will be taken across the five specimens. The experiment is an automated operation in which a 300 gram weight needle is dropped into the paste sample, measuring and recording the depth of penetration. Initial setting is interpreted when the needle can penetrate 36 ± 1 mm, while final setting time is defined when the needle can only leave a mark on the surface of the sample (penetration < 1mm). The test has no significant relationship with the development of cement properties, but simply defines the time it takes for the paste mixes to harden. This test was performed in order to estimate the time, after installation, that the cement at the interface would effectively seal itself off from further water migration; as argued will be the case in Chapter 2. It was established that an average time of 3 hours was required for a Portland cement paste to harden.

3.7.3 Capillary Sorption

This test was carried out on cylindrical samples following the guidelines set in ASTMC1585-04. Control samples mixed to a w/c ratio of 0.5 were also cast, however these were water cured in a tank set to 20°C ± 2°C until ready for testing at 1, 3, 7, 14 and 28 days. At the appropriate curing age the samples were removed from the tank and dried in an oven at 105°C until constant weigh was achieved over a 24 hour period. The cement samples under investigation, were in direct contact with a normally consolidated kaolin clay within the compression rig (refer to section 3.6.3). The samples were in constant contact with the clay until ready for testing at 1, 3, 7, 14 and 28 days.

All specimens were covered in wax around their perimeter, with a plastic coating placed on top. This was done to eliminate the possibility of the water (absorbed through the bottom surface); leaving the sample through the top or sides, i.e. diffusion was restricted to one directional movement. This was done in order to maintain the accuracy of the results.

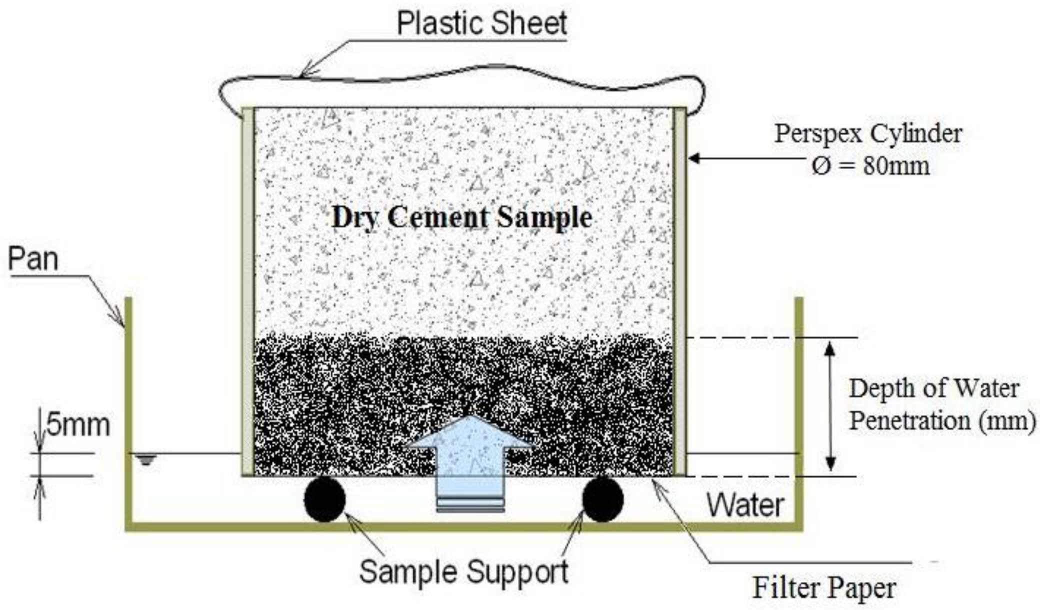


Figure 3.25: Capillary sorption test method and apparatus

The test was performed in order to emphasise that cement has the ability to continue to absorb water even after hardening has taken place, as a result of the tensile forces being generated in the capillaries. The test involved taking periodic measurements up to

6 hours, with a graph of increase in measured mass plotted against square root of time; which allows the sorption to be determined. The initial mass of the sealed cement specimen was measured to the nearest 0.01g, before being placed into 2-5 mm of water. The samples were then weighed at appropriate time intervals, with the results being used to study the level of adsorption. The results should indicate the continual ability of the hardened cement samples to absorb water, even if the rate of absorption is reduced with increased curing time. A further test involving powdered Portland cement was also carried out to determine the potential sorptivity of dry cement during the first 6 hours of it being placed in contact with water.

3.7.4 Mercury Intrusion Porosimetry (MIP)

Mercury Intrusion Porosimetry (MIP) is a method used to determine the pore size distribution within hardened cement paste. The method involves placing a sample into a chamber which is evacuated; mercury is introduced to the sample and is then placed under increasing hydraulic pressure. This pressure forces the mercury into the pores on the surface of the sample; if the pore system is not continuous then the mercury will penetrate the sample by breaking through the pore walls.

MIP is based on the premise that a non-wetting liquid (one having a contact angle greater than 90°) will only intrude capillaries under pressure. The relationship between the pressure and capillary diameter can be represented by (Abeel, 1999):

$$P = -\frac{4\gamma cos\theta}{d}$$

Where: P = pressure, d = diameter of capillary, θ = contact angle of liquid, γ = surface tension of the liquid

MIP tests were carried out on hardened cement samples cured in a normally consolidated kaolin sample for 3, 7, 14 and 28 days. The intention was for these results to agree with the TGA results and confirm the microstructure of the cement paste is continually developing with increased contact time.

The reliability of this method is still debatable as different techniques provide different results. One reason for the unreliability in results lies within the limits of the equipment in place, as the intrusion of mercury is dependent on the capacity of the pressuring device. The equipment used in this investigation is capable of inducing 430MPa of pressure, which falls short of the pressure which causes solidification to the mercury.

Also the fluid in the pores of cement paste is not water, but highly concentrated potassium and sodium hydroxide solutions. Drying of the hardened cement paste, which is needed prior to performing these tests, does not completely remove all traces of these solutions. This leads to some pores, known as "choke points", not being available to the Mercury during testing and as a result underestimates the pore size distribution through the sample.

Paste was dried at a constant temperature of 105^{o}C to remove all water from the sample pores and hopefully allow mercury to fully penetrate the sample. The danger surrounding this technique is the likelihood of microcracking, which will not allow a realistic analysis of the pore development and structure to be conducted.

3.7.5 Nitrogen Adsorption

Nitrogen adsorption is an alternative technique to MIP which is again used to determine the pore structure of paste. This was also performed at 3, 7, 14 and 28 days. The test involves monitoring the adsorption of nitrogen into the sample surface under increasing pressures, which allows the surface of the internal pores and the surface area to be determined.

Nitrogen adsorption works under the assumption that single layer adsorption on pores occurs (internal surface). The test involves placing the sample into a vacuum and dosing the sample with nitrogen gas under increasing pressures. The quality of nitrogen on a surface is measured over a wide range of relative pressures at constant temperature. As the pressure increases the nitrogen is 'forced' into pores and similarly as the pressure is lowered nitrogen is released from the surface of the sample.

This technique has also been viewed as unreliable due to the varying measurements of nitrogen surface area recorded from different researchers. One explanation for this is that the nitrogen particles are larger than water particles therefore cannot penetrate pores with small openings, "Bottleneck Theory."

3.8 GRANULAR UNHYDRATED CEMENT PARTICLES

Prior to testing, the concern was that the central core of a dry cement inclusion would have no access to pore water; due to the inclusion effectively stopping water ingress once the interface had sufficiently hardened. This would mean that the central section of the inclusion would have no involvement in improving the properties of the surrounding soil, thus the dry inclusion method would not be an effective or viable solution for stabilising soils. This concern was generated from the observations made in the Literature Review, most noticeably (Larsson, 2005) where 14 out of 32 'dry inclusions' cast still consisted of dry powdered cement at their core. Similarly to this research the inclusions were left to interact with the pore water in the surrounding soil with no form of mechanical mixing taking place.

For these reasons, a side project was developed which aimed to ensure that the pore water was able to penetrate to the centre of an inclusion. It was decided that the best way to ensure water penetrated the full cross-section of an inclusion was to increase the permeability and function more like a vertical drain. The concept of generating granular particles of dry unhydrated cement was developed and originated from an idea to combine the dry inclusion method of this study, with that of the established sand drain technique.

3.8.1 Theory

The intention was for dry unhydrated cement granules to be created; which would aim to satisfy their natural affinity for water and absorb water from the clay soil[1].

Similar to the 'dry cement inclusion' this absorption would cause an immediate dewatering effect to be experienced by the surrounding soil and a reduction in pore pressure would be experienced in a zone immediately surrounding the inclusion. This reduction in pore pressure would induce a pressure gradient within the soil, with water being drawn towards the inclusion. However, a more long-lasting dewatering effect would also be present with these cement granules, as once the granules have absorbed

[1] This absorption would continue through the granules until all cement particles, even those at the centre of the granule, were interacting with water.

the level of water required to satisfy the cements demands their function would be similar to that of a sand drain.

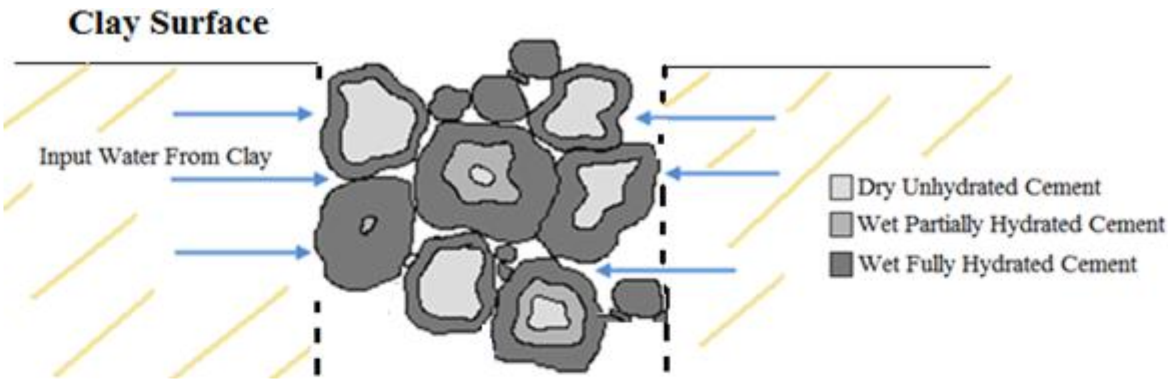


Figure 3.26: Intended dewatering system with dry unhydrated cement granules

3.8.2 Materials and Methods Adopted to Create Dry Unhydrated Cement Granules

Two materials were selected to form these unhydrated cement granules. These materials were selected based on the quantity of water present in their chemical properties, as water is the reactant with which cement hydrates. It was essential the cement granules were unhydrated prior to testing as this characteristic separated the theory from that of an established sand drain. The materials and methods undertaken to form the granules are discussed.

Material Choice 1 - Decane ($CH_3(CH_2)_8CH_3$)

The first attempt to form granular unhydrated cement grains involved a mixture combining Decane, Xanthan gum and 600μm sieved Portland cement. Decane was initially selected as it is an anhydrous solution consisting of 0.005% water, by weight, which would ensure that little, if no, hydration would occur in the cement grains. Powdered Xanthan gum was selected as it is a standard food additive and is often used whenever a gel-like quality is sought. The intention was for Xanthan gum, when mixed with liquid Decane, to form a gel capable of adhering to the dry cement grains. The mechanical process of mixing would then cause the cement grains to combine and produce a granular material; which has not undergone any form of hydration.

This was successfully achieved with a mix consisting of 5% Xanthan gum, by weight of liquid Decane, where mixing was performed in a standard mixing bowl by hand, for a period of 10 minutes, until a consistent sticky gel was achieved. Once the gel was generated Portland cement, twice the volume of the Decane by weight, was incorporated into the mix. The sample was mixed for approximately 10-15 minutes until no loose cement was visible in the mix.

The mixture resulted in a granular material product of reasonably sized particles (typically 4-5mm). However, the granules were not satisfactory for the purposes of this investigation, as they were easily broken and disturbed when handled. As these materials would require transportation to site it was clear this would not be achievable with this mixture. Several mixtures comprising of different portions of liquid Decane, by weight of dry cement, were performed with the same outcome recorded for each mix. The decision was taken to substitute Sodium Silicate (water glass) in place of Decane.

Material Choice 2 - Sodium Silicate (water glass)

Sodium Silicate was deemed as an appropriate replacement for Decane as it is often used to increase the binding properties of various products. The Sodium Silicate was supplied in powdered form which did require the addition of water. However, it was hoped that the small amount of water necessary to produce the jellylike liquid would crystallise during initial mixing i.e. would not be available to start hydration on contact with cement. Xanthan gum was again used to aid with the formation of a gel-like substance, and was in powder form so would utilise some of the water added to the mix.

Sodium Silicate and 5% Xanthan gum, by weight of Sodium Silicate, were initially mixed with water (30% volume by weight of dry sodium silicate was found to be the minimum to achieve a gel-like consistency). Mixing was performed by hand until a gel was formed. A great amount of heat was experienced during initial mixing as a result of the sodium silicate reacting with the water. Mixing was performed for 10 minutes with the gel increasing in thickness with increased mixing. At this stage the Portland cement (50%, 75% and 100% by weight of powdered Sodium Silicate) was added to the mix. Mixing was then transferred to a standard mixing machine where continual mixing was performed on the mixture for approximately 30 minutes.

The resultant products from each mixture were solid granular particles of varying size and shape (typical range from 1 to 17mm in diameter). However, for all mixes, it was clear that hydration had been encountered as a result of the water in the mix, as the granules were visually dark and dense. TGA testing on various granules from each of these mixes confirmed hydration had taken place.

Tests, involving granular particles from each mixture being submerged in water for a constant 72 hours, were carried out in order to determine if water absorption was still possible; given the dense state of the granules and the knowledge that hydration had already taken place during the mixing procedure. The results showed that granules from the mix involving 100% Portland cement by weight of Sodium Silicate were capable of absorbing 9% water by weight of dry mass. The granules from the 50% and 75% Portland cement samples showed water absorption of 2% to 6% respectively. This technique was therefore considered unsuccessful in achieving the aims set prior to proceedings, as little water absorption is capable of taking place in comparison to a dry cement inclusion.

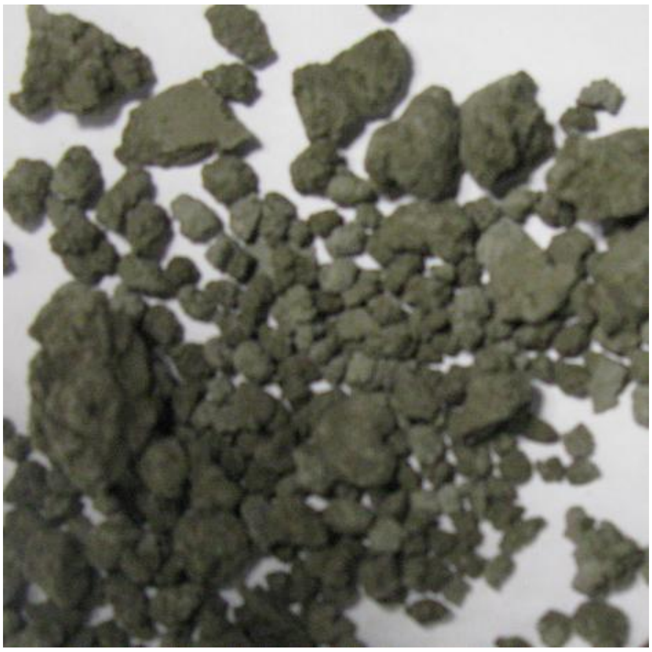


Figure 3.27 Granular particles obtained from Sodium Silicate, Xanthan Gum, water and Portland cement

3.8.3 Outcome of Granular Cement Experiments

Having spent a lot of time discussing other possible materials with suppliers - which could achieve granulation (without causing hydration), the decision was taken to suspend work and concentrate on the dry powdered inclusion. This decision was based on a lack of material options and the time constraints on the project. As a result no further mention of granulated cement will be presented in this study.

3.9 CHAPTER SUMMARY

This Chapter described the materials, equipment, test standards and procedures adopted by the Author in order to successfully deliver the aim and objectives defined in Chapter 1. The following can be summarised:

i. Kaolin was used to represent clay soil as it has a high plasticity and can be mixed to a range of moisture contents. Arbitrary inclusion diameters of 38 and 70mm were tested at 1, 3, 7, 14 and 28 day curing periods and a 100mm inclusion tested solely at 28 days, with Portland cement (to BS EN 197: CEM 1) being used to form the inclusions.

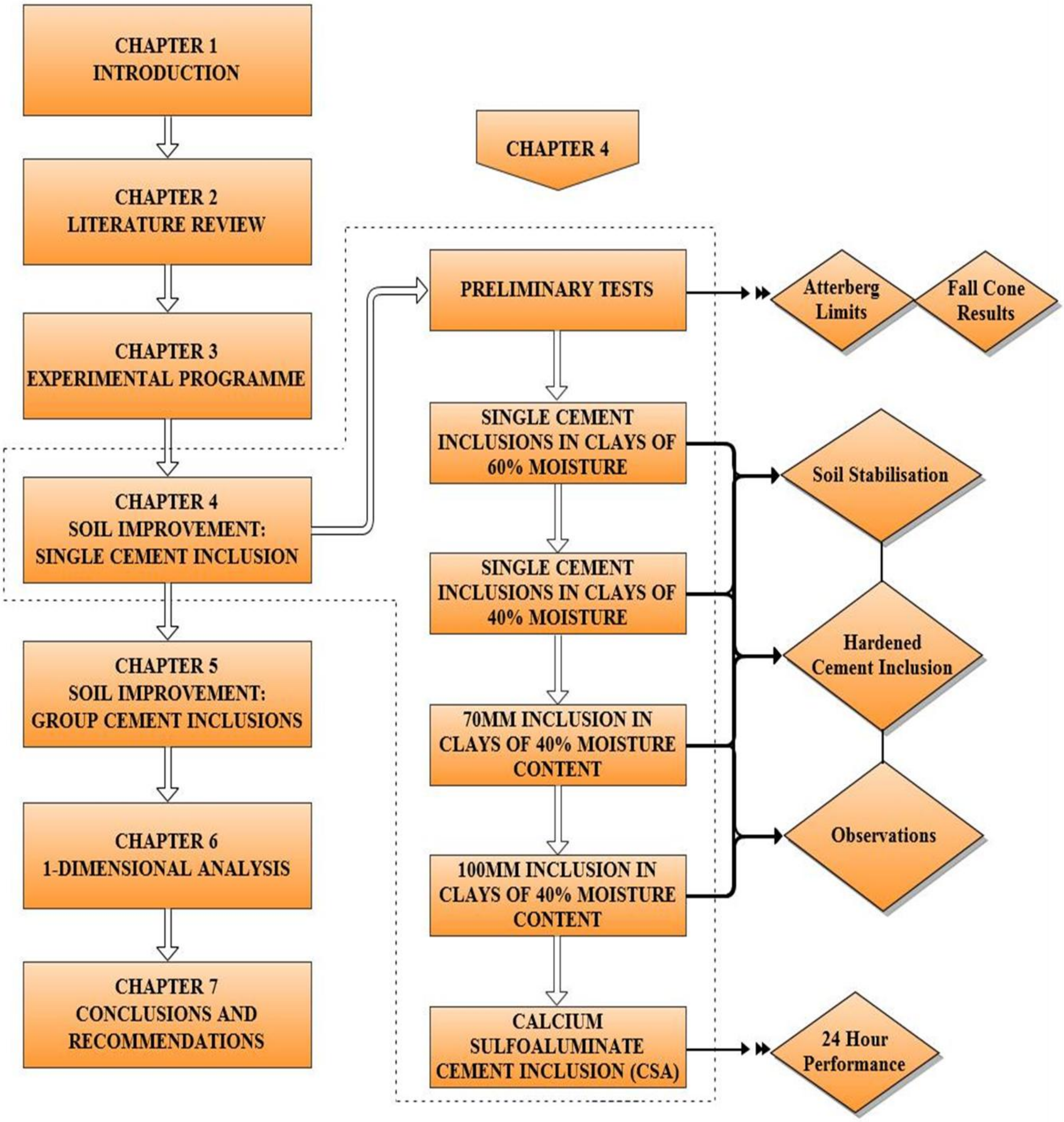
ii. Fast setting CSA cement was tested and compared to traditional Portland cement 24 hours after installation, in order to determine if rapidly dewatering the soil has any benefit to the shear strength improvement experienced in the surrounding clay.

iii. Using the Fall Cone Test and values obtained from the Atterberg Tests, the liquidity index was correlated to a corresponding shear strength representative of the kaolin clay used in this programme of work. This is an adaption to the relationship of liquidity index and shear strength presented in Atkinson (2007).

iv. Moisture content samples and shear vane tests were performed to determine the strength of the kaolin clay, with TGA used to monitor the progress of water migration through the inclusion to the cement at the core. Filter Paper Suction and Capillary Sorption Tests were also be performed, to determine both the ability of the cement inclusion to draw in water once cement hardening had taken place and the suctions experienced in the soil as a result of introducing an inclusion of dry cement.

v. Normally consolidated kaolin clay samples (preconsolidation pressure 140kPa) were tested in the 1-D compression rig at 1, 3, 7, 14 and 28 days, with TGA, MIP and Nitrogen Adsorption tests performed in the cement inclusion and moisture tests performed on the clay. Overconsolidated samples (OCR = 1.5 and 2.0 respectively) were also tested after being in contact with a cement layer for 7 days in the rig.

vi. An attempt to create granular non-hydrated cement particles; which have the capacity to absorb water and dewater the soil in a similar manner to sand drains was also carried out. Using two separate materials – Decane and Sodium Silicate, along with Xanthan gum and Portland cement. These attempts were not successful and will have no further mention in this investigation.



“In order to succeed you must fail, so that you know what to do the next time”

Anthony J. D’Angelo (1972-Present)

CHAPTER 4

SOIL IMPROVEMENT:

SINGLE CEMENT INCLUSIONS

4.1 INTRODUCTION

In this Chapter the results from hand mixed kaolin clay samples stabilised by the introduction of a single dry cement inclusion are presented. This Chapter discusses the radial influence of a single inclusion with respect to dehydration of the surrounding clay soil; in order for the cement to hydrate. Two clay conditions, of initial moisture contents 40% and 60% respectively, were under investigation, with the hydration performance of the inclusion itself also being assessed. The cement samples were kept in contact with the clay soil for 1, 3, 7, 14, and 28 days, as these are common curing periods adopted for monitoring the strength gain of concrete samples. Kaolin clay samples were prepared as discussed in Section 3.4.2, with moisture content extractions, shear vane tests and Thermogravimetric testing (TGA) all performed in this section of the testing programme.

4.2 PRELIMINARY TESTS

4.2.1 Atterberg Limits

Atterberg Limits tests performed in accordance with BS 1377-Part 2: 1990, allowed the following limits of the kaolin clay used in this investigation to be determined:

Table 4.1: Atterberg limits for kaolin clay

Atterberg Limit Test Values	
Plastic Limit, %	28.5
Liquid Limit, %	75.0
Plasticity Index	46.5

Correlation of Liquidity Index with Shear Strength

As discussed the liquidity index of the kaolin clay was correlated to its performing shear strength with the use of a fall cone test. This test was performed over a range of moisture contents which allowed the relationship between the kaolin clays shear strength and liquidity index to be compared to the relationship provided by (Atkinson, 2007).

At the same time, the decision was taken to perform hand shear vane tests at different moisture contents in order to provide a comprehensive set of results. Unfortunately, due to the hand shear vane not being a sensitive device in soft soil, shear values of clay samples prepared to moisture contents greater than 45% could not be detected on the equipments scale. Another problem was the inability of the Author to obtain homogenous clay samples at moisture contents below 40%. These problems meant that a limited amount of data could be obtained when performing hand shear vane tests.

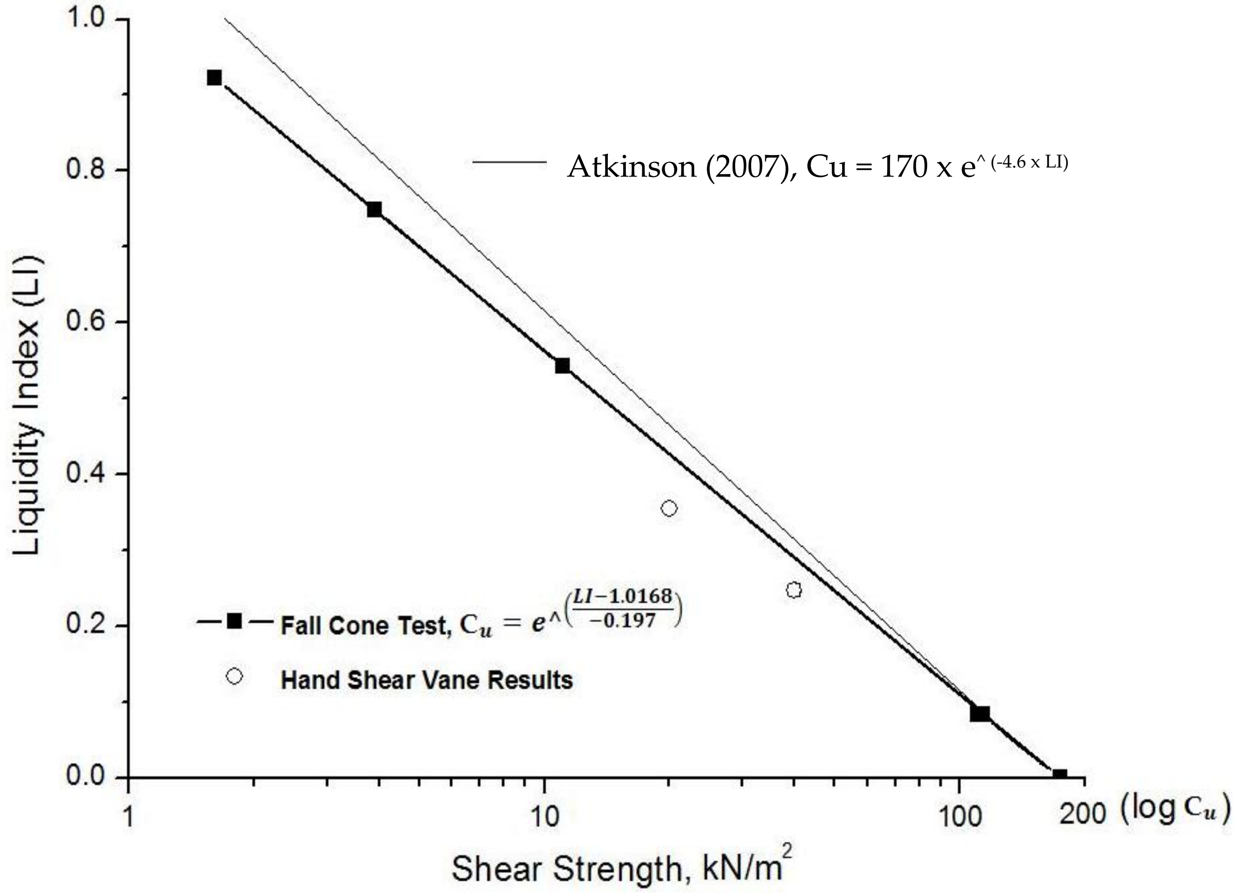


Figure 4.1: Soil shear strength in relation to liquidity index

From the graph, it appears that the trend in results is similar for the kaolin clay used in this project to that proposed by (Atkinson, 2007). However, Atkinson can be shown to overestimate the soil shear strength. The relationship generated from these fall cone test results will be utilised in this study to convert the soils moisture content (hence liquidity index) into a shear strength value.

4.3 SINGLE CEMENT INCLUSIONS IN CLAYS OF 60% MOISTURE

4.3.1 Kaolin Clay Soil Testing

Prior to installing the inclusions, control samples were tested to ensure that 60% moisture (± 1%) was found at all locations in the sample i.e. a homogenous sample was achieved during the hand mixing procedure (described in section 3.4.2). Tests at 60% moisture were initially conducted using dry cement inclusions of 38mm diameter, as this limited the volume of hand mixed kaolin required to perform a test.

The following contour graphs (refer to Figure 4.2) permit a profile of the moisture change and movement throughout a clay sample to be generated, as a result of the dry cement utilising the soils pore water at increasing distances and depths from the inclusion interface. The change in moisture content will be observed with respect to time, by extracting moisture content samples (refer to section 3.5.1) at increasing radial distances from the inclusion-soil interface. It is hoped that these contour graphs will highlight a reduction in moisture content; in comparison to the initial 60%, due to water being removed from the soil in order to initiate hydration of the cement in the inclusion. The magnitude and distance to which a reduction in moisture is observed from the inclusion interface will highlight how successfully the proposed system has improved the surrounding soils strength through consolidation. This should be possible as these moisture values can be used to determine the liquidity index, which had already been correlated with the soil shear strength (refer to Figure 4.1) using the fall cone test results.

A point to note about the graphs in Figure 4.2, is the absence of a 0,0 coordinate at the top left hand corner. This is the result of the moisture content being taken at the central position of the 10 x 10 x 10mm samples, thereby the first moisture content is actually at 5mm from both the inclusion interface and top surface of the clay sample.

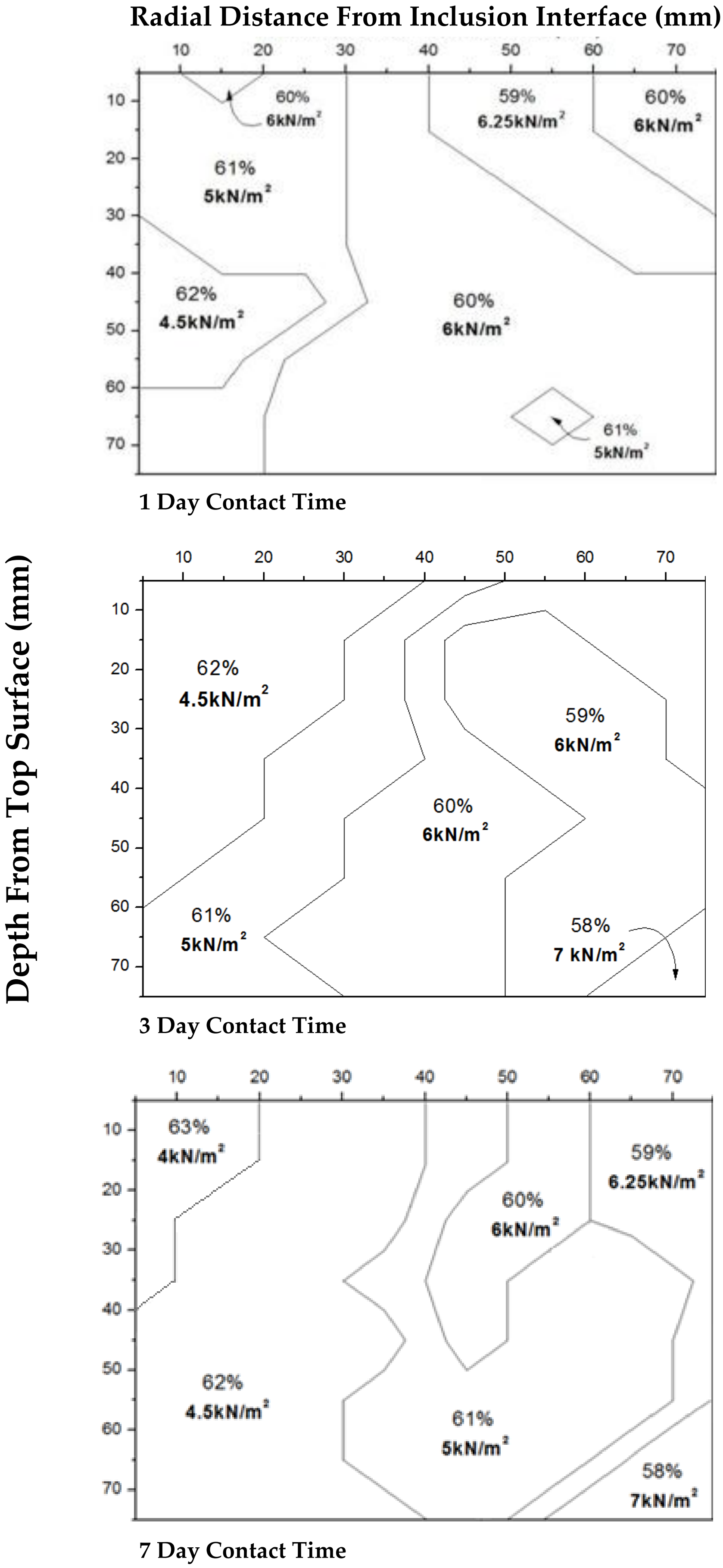
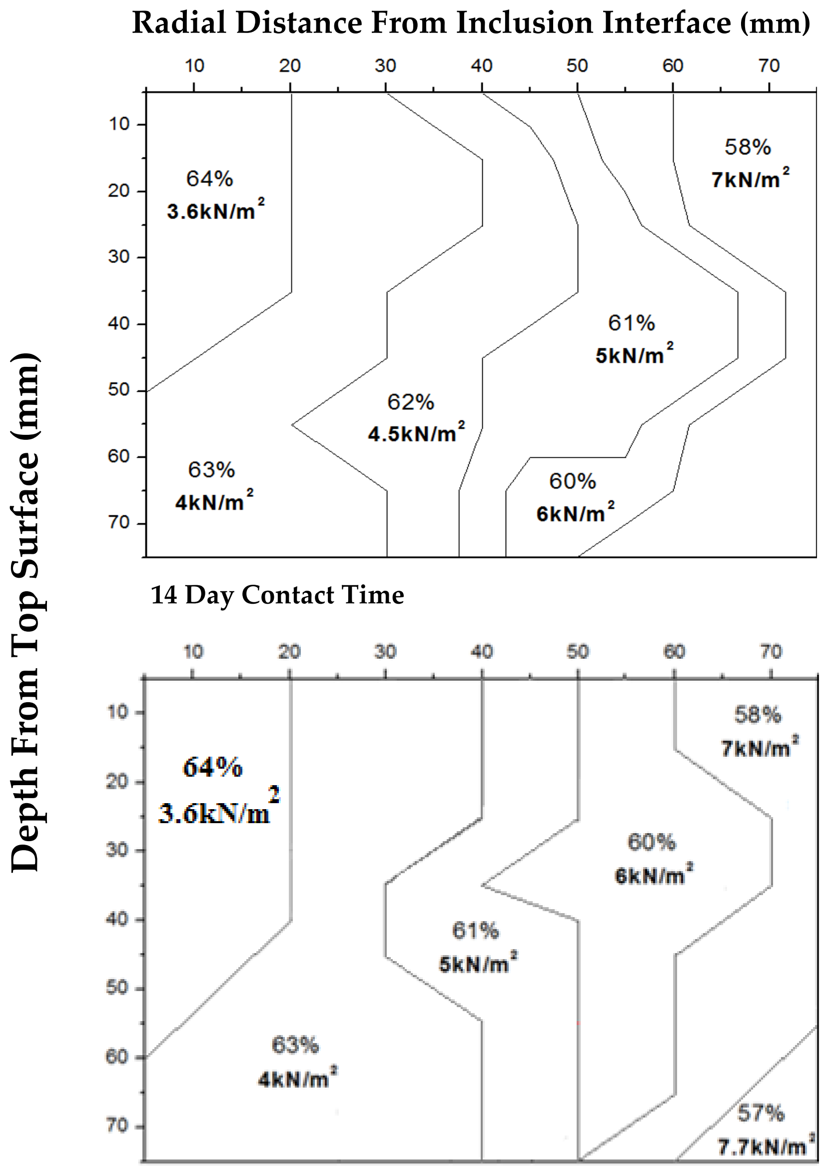


Figure 4.2: Radial moisture movement with increased contact time (Cu = 6kN/m²)



28 Day Contact Time

Figure 4.2: contd.....

Summary of Graphs

The graphs produced in Figure 4.2 are useful for portraying the moisture changes experienced in laboratory prepared kaolin clay of 60% moisture, as a result of the cement inclusions utilising the pore water up to 28 days. However, in order to summarise the data from each of these contour graphs, it was decided that the average moisture content at 10mm intervals from the inclusion interface should be taken and plotted on one graph; for each test time. As the moisture content at each 10mm interval can be seen to change quite drastically over the full length of the inclusion, it was necessary to split the inclusion into three sections as detailed in Figure 4.3, in order to portray a more accurate account of the moisture changes occurring in the soil over time.

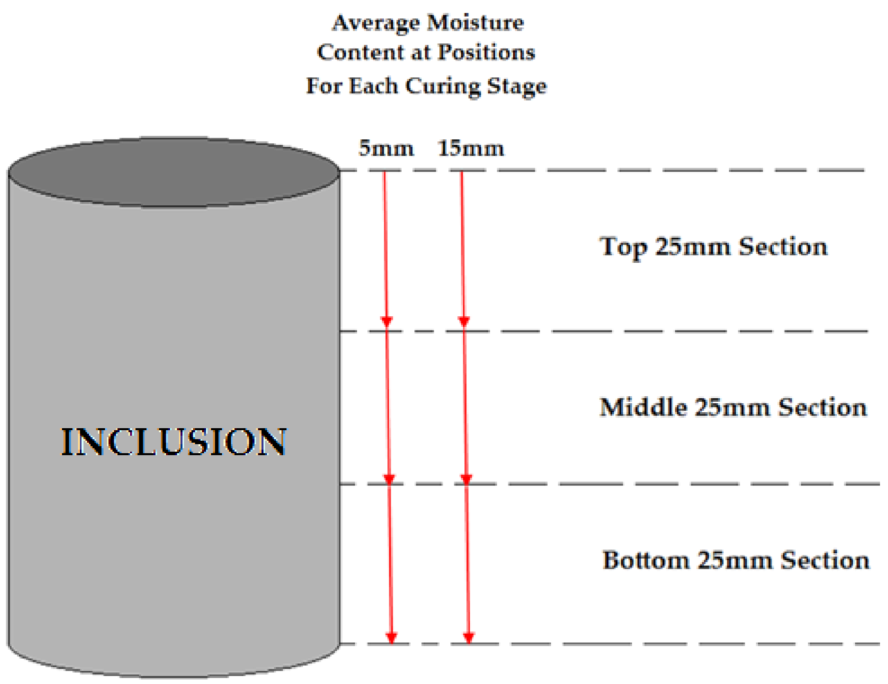


Figure 4.3: Breakdown of sections to summarise moisture changes

By performing this task the following graphs were produced:

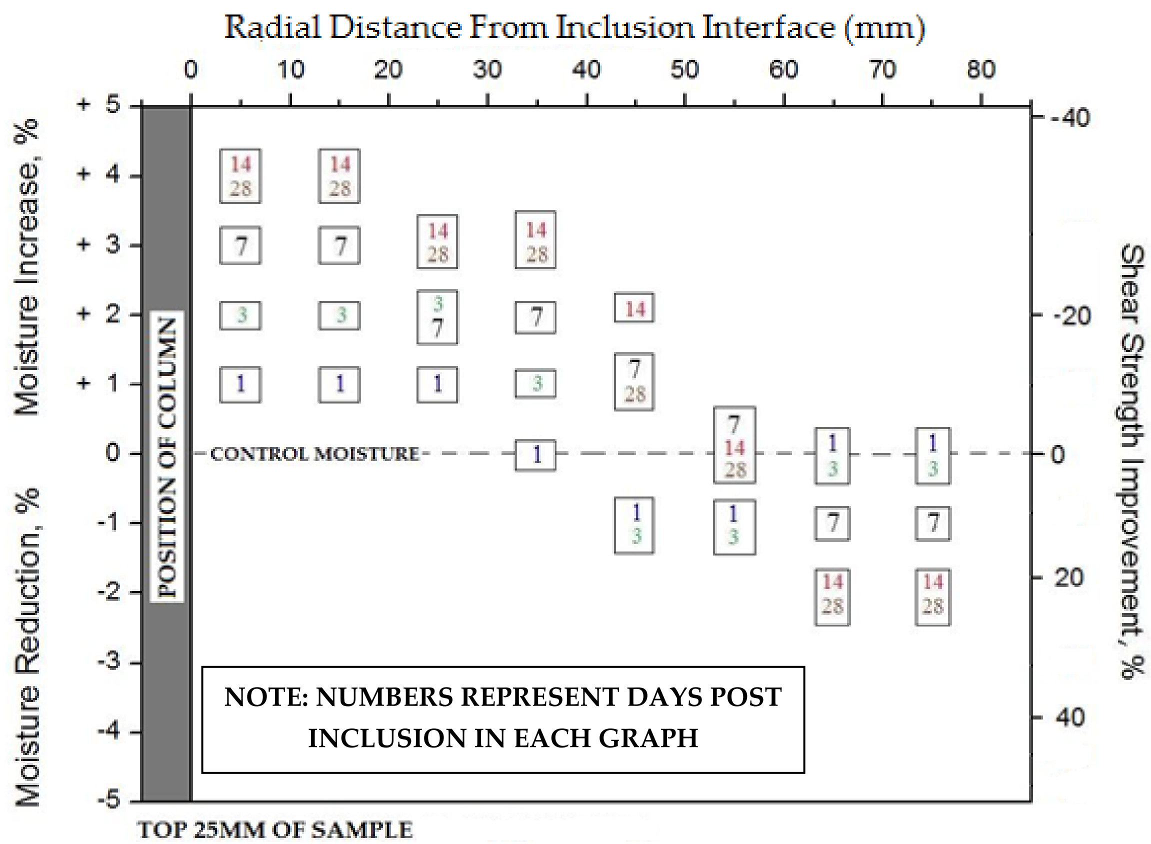
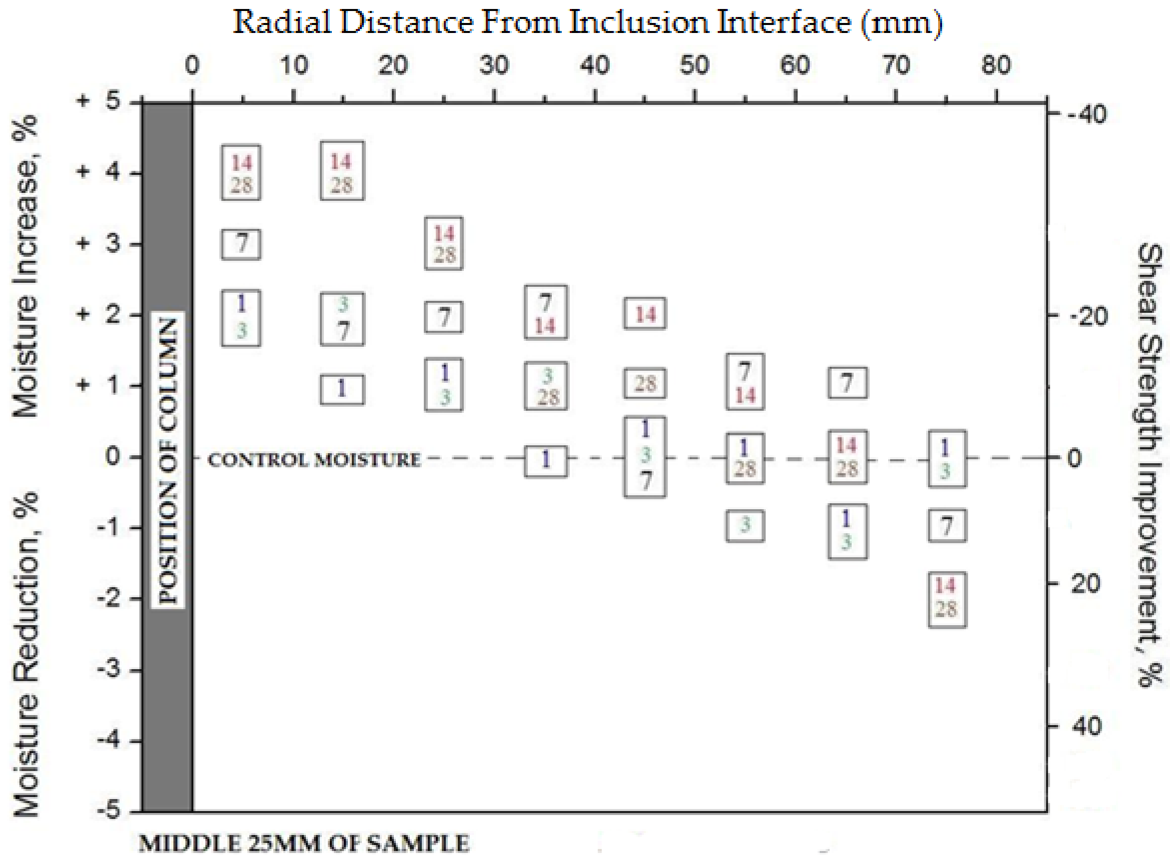


Figure 4.4: Summary of the changes in moisture content experienced over time in a clay soil of 60% moisture



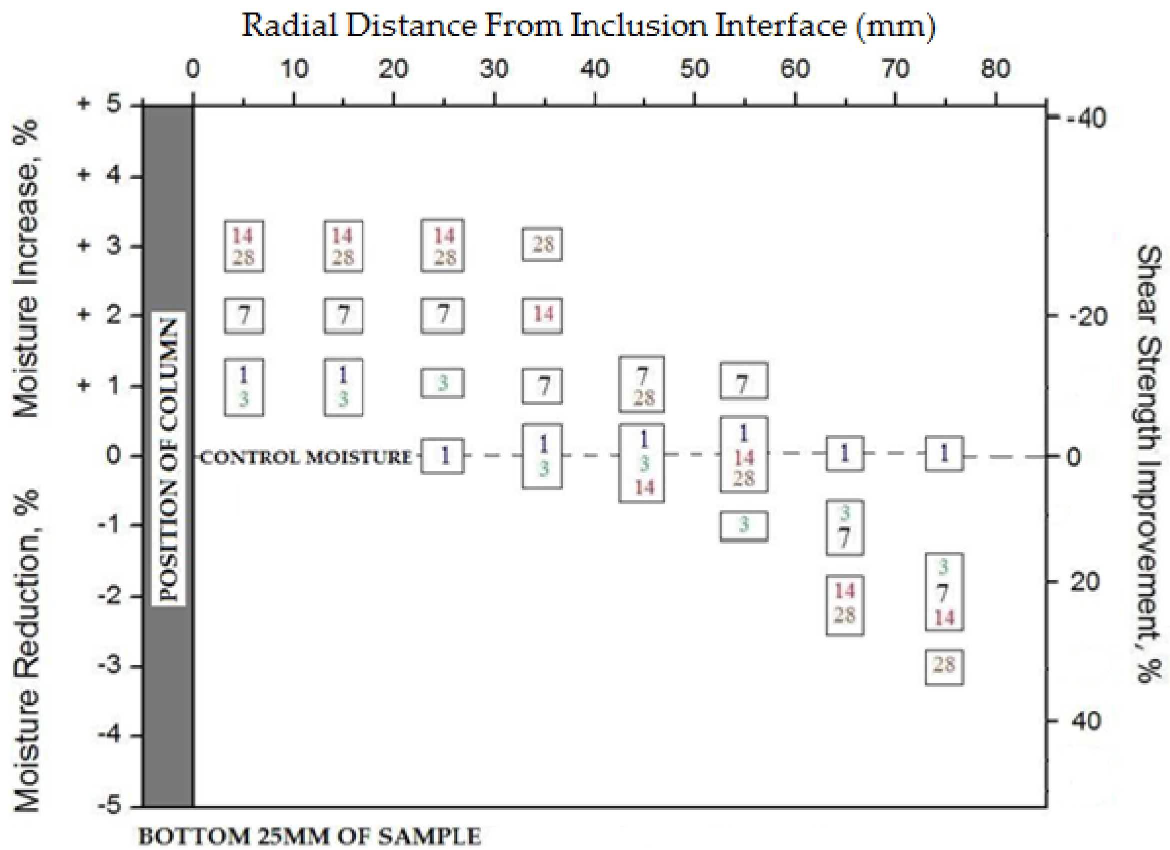
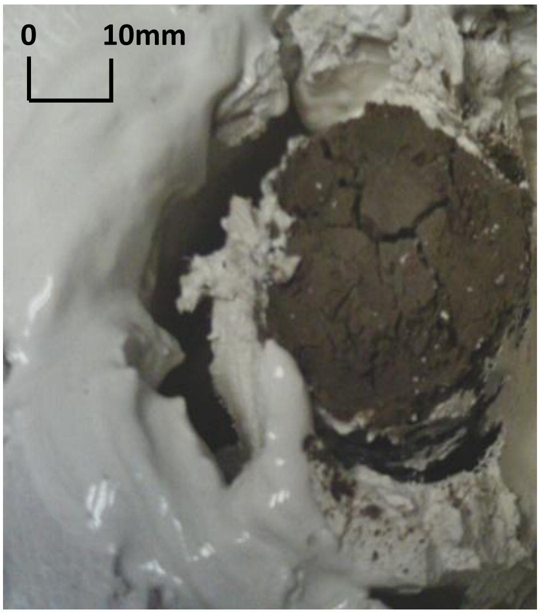


Figure 4.4: contd..............

It is clear from these graphs that the introduction of a dry cement inclusion has an adverse effect on the shear strength of the 60% clay soil in a zone immediately surrounding the inclusion interface, due to the increased volume of moisture surrounding the soil-inclusion interface. It was anticipated that the dry cement would absorb water immediately surrounding the inclusion, resulting in reduced moisture contents and increased soil shear strengths being recorded to some radial distance (x).

However, upon extracting samples, the Author noted how the soil appeared to be extremely soft in comparison to the control and it became extremely difficult to obtain sample extractions. The Author also identified large pools of water surrounding the inclusion, which supports the increased moisture values present in the results.



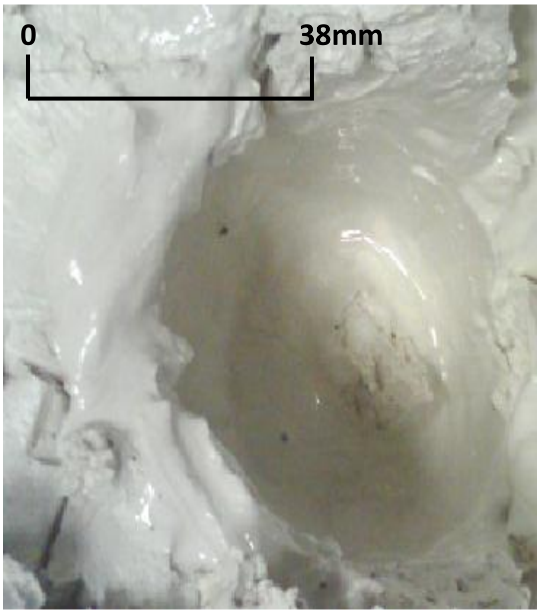


Figure 4.5: Pool of water surrounding the inclusion at 3 days

The volume of moisture encountered at the soil-inclusion interface increased with increased contact time, and was observed to extend to greater radial distances. This is reflected in the results, with an increased reduction in strength being recorded at greater distances from the inclusion with increased time; with the largest reductions recorded at the interface. The Author has accredited this to two separate actions; the low response rate of clay and cement hardening; both of which are discussed in the following.

Initially, the pore water occupying the space between the soil particles is in a state of equilibrium; this is in accordance with the Principles of Effective stress. As the dry cement inclusion is introduced, water is instantly drawn from the soil by absorption as the dry cement attempts to satisfy is natural affinity for water. This causes the water in the soil; in a zone immediately surrounding the inclusion, to reduce[2] and a pressure gradient to form. The pore water in the soil at increasing radial distances is then drawn towards the inclusion under this pressure gradient.

However, due to the low permeability of the clay soil, it will take time for this information to be received at some radial distance (x) from the interface, i.e. apart from the clay immediately surrounding the soil-cement interface, the rest of the clay soil is oblivious to the fact that there is a boundary at which drainage has started to occur (Muir Wood, 2009). Still as the information reaches soil at increasing radial distances, more and more water begins to migrate towards the inclusion. This partly explains the increasing volume of water being encountered near the soil-inclusion interface with increased curing time. However, cement hardening also plays a leading role.

The initial absorption of water into the inclusion causes a concentration difference, between the newly wetted cement and the dry unhydrated cement at the core, to be induced in the cement. This causes the transport mechanism of water into the cement inclusion to change from absorption to diffusion. Water diffuses through dry cement in accordance with Fick's Law and causes a hydration reaction to occur. This reaction causes the cement to harden over time. It also reduces the inclusions permeability as a result of calcium aluminate hydrate and/or calcium silicate hydrate (C-S-H) gels forming, and subsequently crystallising, in the void spaces. Paste setting tests were carried out prior to testing. The results indicated that Portland cement pastes mixed to a w/c ratio of 0.5 take on average 3 hours to harden. After this period the interface of the soil-cement inclusion would effectively harden and the mechanism of water transport would ultimately change from diffusion to capillary suction (as a result of surface tension acting in the newly formed capillaries). This mechanism change would limit ingress of water and would explain the excessive moisture build up experienced around the inclusion.

[2] Thus the soil-column interface can best be described as a drainage path.

Therefore, as a consequence of the clay soils poor response time, water migrating from large radial distances reaches the inclusion after cement hardening has taken place. The volume of water migrating towards the inclusion is likely to be far greater than the volume of water being absorbed by the inclusion once hardening has taken place. As a result water is consequently gathered around the inclusion perimeter and a reduction in soil shear strength occurs.

This does not support the theory that dry cement inclusions will improve the soils strength as a result of water being removed from the soil, as in actual fact the inclusions presence causes a reduction in strength to materialise around the perimeter of the inclusion; due to build up of moisture.

However, from Figure 4.4, it can also be observed that as a result of this pressure gradient causing the pore water to migrate towards the inclusion, a moisture reduction (hence strength increase) is observed at radial distances greater than 45mm from the inclusion. Similarly to the moisture increase observed at the inclusion interface, the magnitude of moisture reduction is dependent on the contact time.

4.3.2 Hardened Cement Inclusion Testing

Thermogravimetric Analysis (TGA) was carried out on the hardened cement samples, as described in section 3.7.1, in order to determine if water had the ability to penetrate and react with the cement through the entire cross section of the inclusion. Samples were taken at every 5mm intervals, starting from the soil-inclusion interface through to the core of the inclusion. Tests were conducted at 1, 3, 7, 14 and 28 days. This allowed a water penetration/hydration profile to be generated with respect to increasing contact time (Figure 4.6).

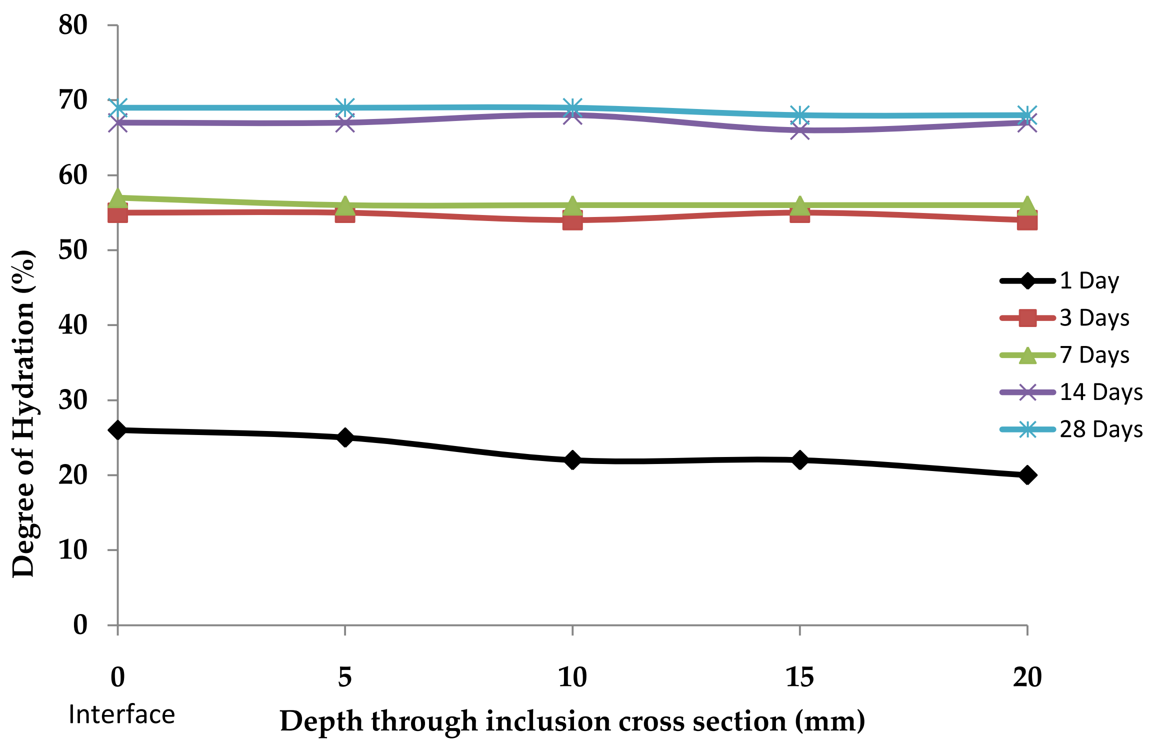


Figure 4.6: Cement hydration throughout 38mm inclusion cross section (60% moisture) with increasing contact time

It was anticipated prior to testing that water would manage to reach the core of the inclusion during the initial diffusion process, however would be subdued after sufficient hardening of the cement at the interface. This would result in a greater degree of hydration being experienced near the interface, in comparison to the core of the inclusion. This would have been correct reflecting the TGA results for the 1 day inclusion, as these indicate that water has managed to penetrate to the centre of the inclusion and react with the cement, however not to the same degree as cement at the interface. However, inclusions cured for periods greater than 1 day show that the degree of hydration at the centre of the inclusion is the same as at the interface, which suggests the core has the same level of access to water as the cement at the interface. This was not expected.

Possible reasons for these results could be explained by the large tensile cracks observed by the Author at the top of each of these inclusions (as shown in Figure 4.7). These cracks were visible throughout the entire cross section and could explain why the centre of the inclusions hydrated to such a large degree; as tensile cracks would provide easy access for water to penetrate and react with the cement at the core. In other words, the inclusion would function in a similar manner to a vertical drain.

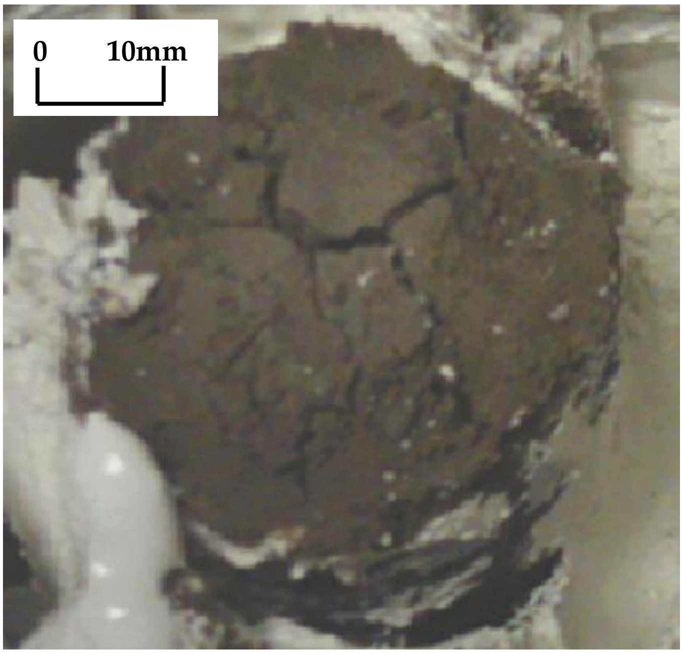


Figure 4.7: Tensile cracks through 38mm inclusion cross section

These cracks were observed to increase in size with an increase in contact time which is most likely a consequence of the volume changes experienced by the inclusion during hydration; also known as shrinkage. Shrinkage results from the chemical reaction of the cement with water, which produces hydration products of less absolute volume than the initial reactants. These volume changes develop tensile stresses between the inclusion and clay, which result in cracks when this tensile stress exceeds the tensile strength of the inclusion. The level of shrinkage experienced in this case is most likely due to the rapid hydration of the cement. However, further studies on the shrinkage of these inclusions would have to be conducted, as this is out with the scope of this project of work.

Each inclusion sample was weighed prior to and after installation, in order to obtain a rough estimate of the volume of water being absorbed by each inclusion. By doing this 40% water, by weight of dry cement, was shown to be absorbed during the first 24 hour period after installation. This suggests that a great deal of water is absorbed during the initial stages of the cement inclusions coming into contact with the clay. All samples after the first 24 hours of curing were shown to absorb approximately 50-54% of water, by weight of dry cement, which suggests that water absorption continues to take place even after the cement at the interface has sufficiently hardened. However, this level of absorption could be a consequence of the large tensile cracks present in the inclusions cross section, which would allow the water to freely move into the centre of the inclusion. It might not be a true reflection of a dry cement inclusions performance in the absence of these tensile cracks. In order to determine if tensile cracks play a significant role or not, further testing will be required. For this reason, the Author will perform an analysis on clay soils mixed to an intended 40% moisture content, as this should reduce the level of shrinkage and consequently the formation of tensile cracks observed in this case.

Another physical observation recorded by the Author involved necking at the lower end of the inclusion. This would be the result of the surrounding clay entering the inclusion after installation, as the initially dry unhydrated cement would be unable to resist the lateral thrust from the surrounding clay soil. This could potentially lead to an uneven level of improvement being provided to the surrounding soil along the full length of the inclusion which is a problem associated with the lime-cement column method.

4.3.3 Conclusions Drawn From Analysis of a Single Inclusion at 60% Moisture

In this case, it would appear from both the Authors own observations and the results produced from moisture content samples, that incorporating a dry cement inclusion is not a viable solution for stabilising clay soils with an initial moisture content of 60%. It is the Authors views that although the cement is shown to continually absorb water from the surrounding soil, the low response rate of clay, coupled with cement hardening, contributes to excess moisture build up around the inclusion. This excess moisture continues to develop with increased curing time, which as a consequence leads to a continual reduction in the soils shear strength being generated around the inclusion perimeter.

An improvement in shear strength was recorded at increasing radial distances from the inclusion and is a direct consequence of the pressure gradient set up within the sample; as water is drawn towards the inclusion, the clay at some radial distance experiences a reduction in water content. This will continue until equilibrium is restored in the soil sample. However, due to the level of weakened soil surrounding the inclusion, this improvement was not deemed sufficient to say overall improvement to the soil had occurred.

The system was intended to improve the strength and stability of the clay surrounding the inclusion; instead the presence of the dry cement appears to weaken the soil. Therefore, the systems objectives have not been met in this particular case. For this reason, the Author of this report has decided not to continue analysing the effects of dry inclusions whilst working in clays of 60% moisture content or above, as the system appears ineffective in stabilising the soil at such high moisture contents. Instead focus will switch to clay soils of 40% moisture.

4.4 SINGLE CEMENT INCLUSIONS IN CLAYS OF 40% MOISTURE

4.4.1 Kaolin Clay Soil Testing

The ability of the dry cement inclusions to utilise pore water from clay soils of 40% moisture content, was investigated in the exact same manner as the previous 60% clay samples. Again moisture control samples were extracted to increasing radial distances from the inclusions interface and at increasing depths from the top surface of the soil, which allowed moisture movements; as a result of the dry cement inclusions interaction with water, to be monitored. The moisture content results in each sample are depicted in Figure 4.8.

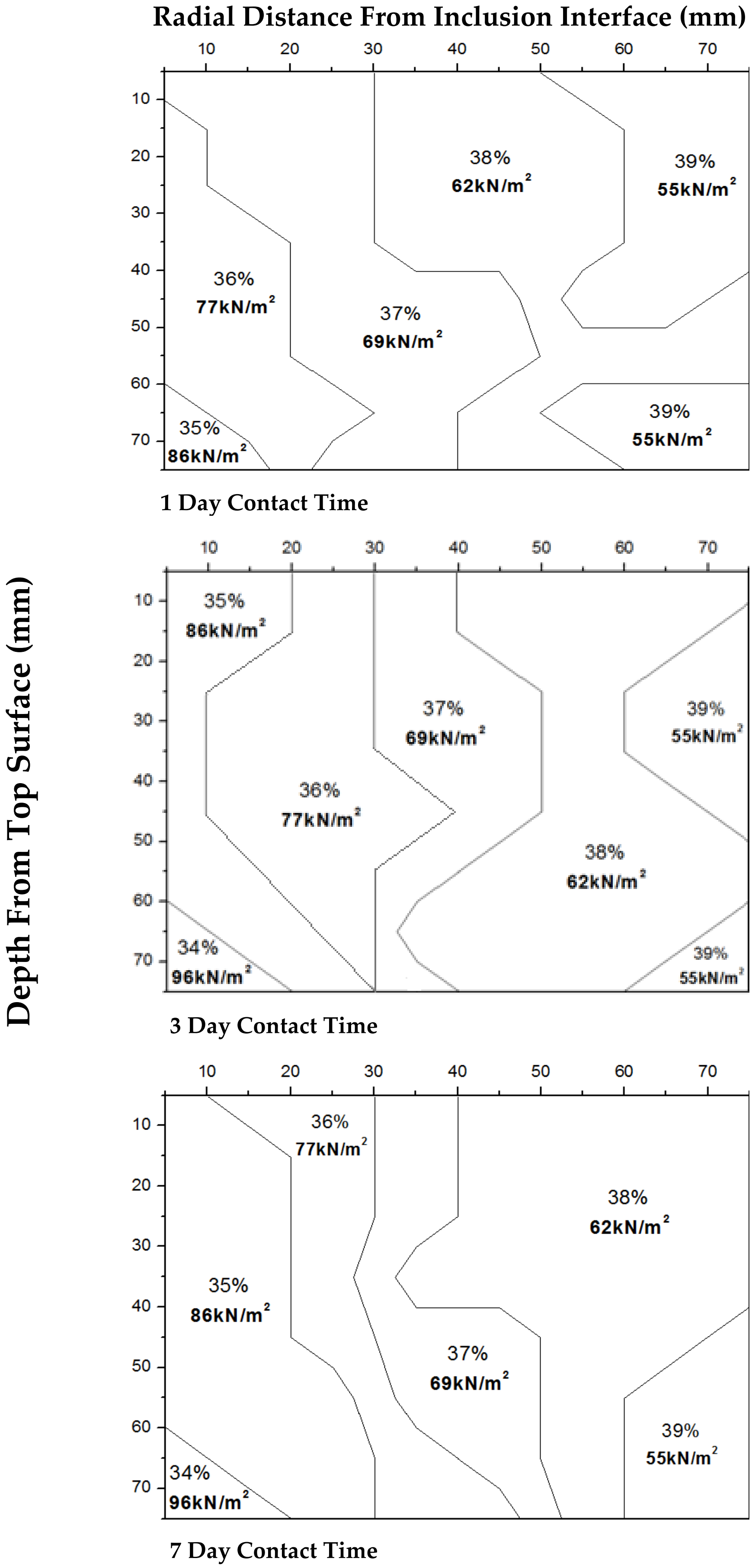


Figure 4.8: Radial moisture movement with increased contact time (Cu=50kN/m^2)

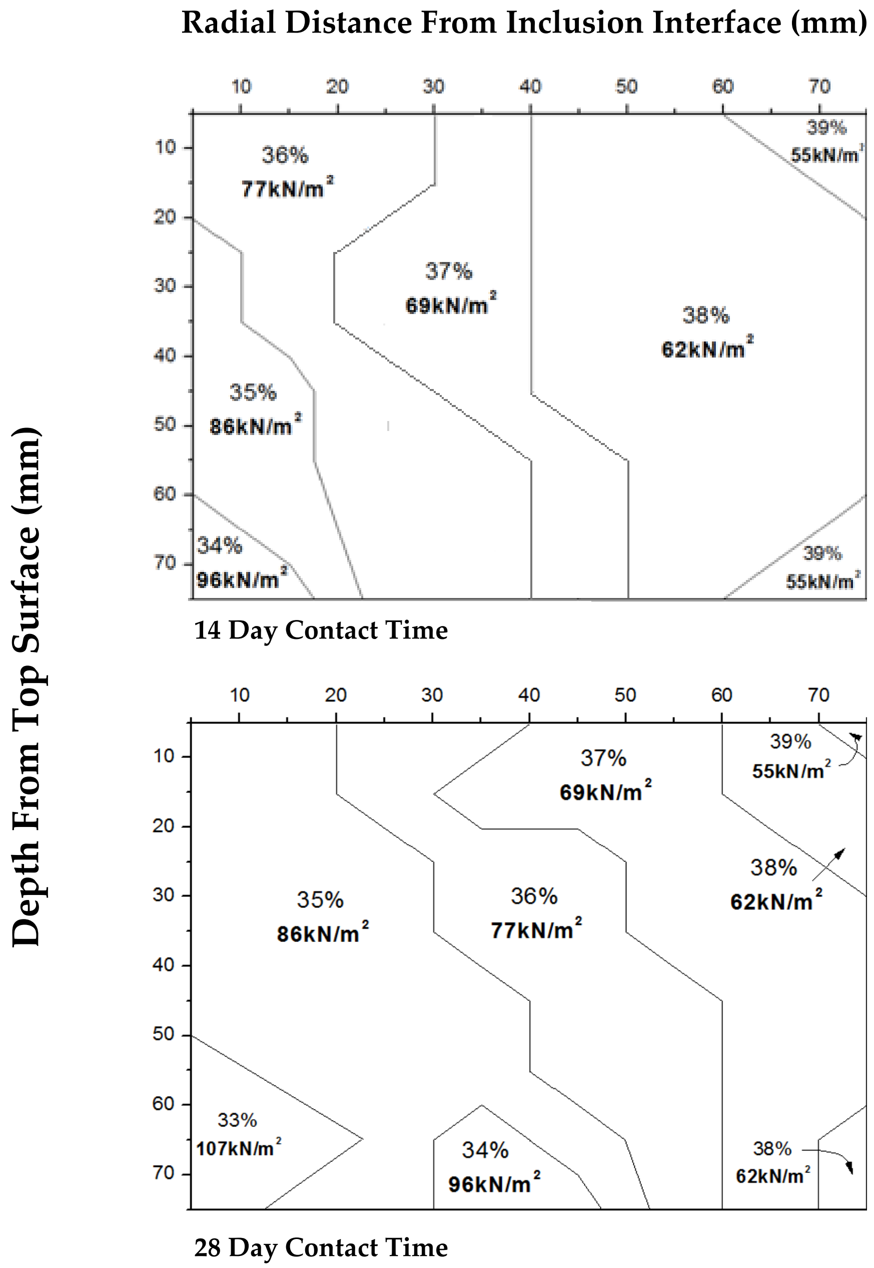


Figure 4.8: contd......

Summary of Graphs

Again the decision was taken to split the inclusions into three 25mm sections along their length (as shown in Figure 4.3), and take the average moisture content at 10mm interval along each section. This allowed the following summary graphs to be produced with the moisture content considered for each stage of curing.

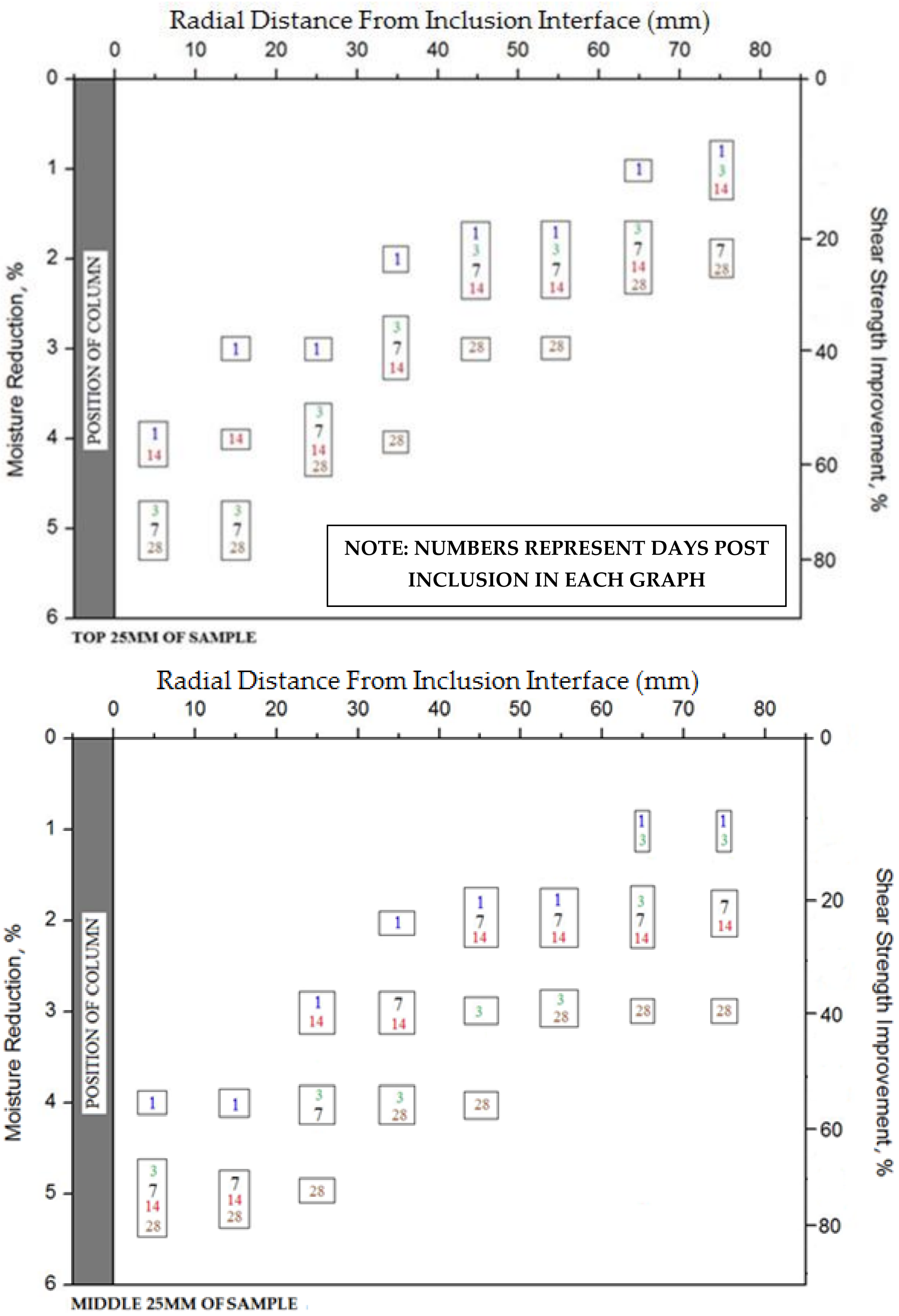


Figure 4.9: Summary of the changes in moisture content experienced over time in a clay soil of 40% moisture

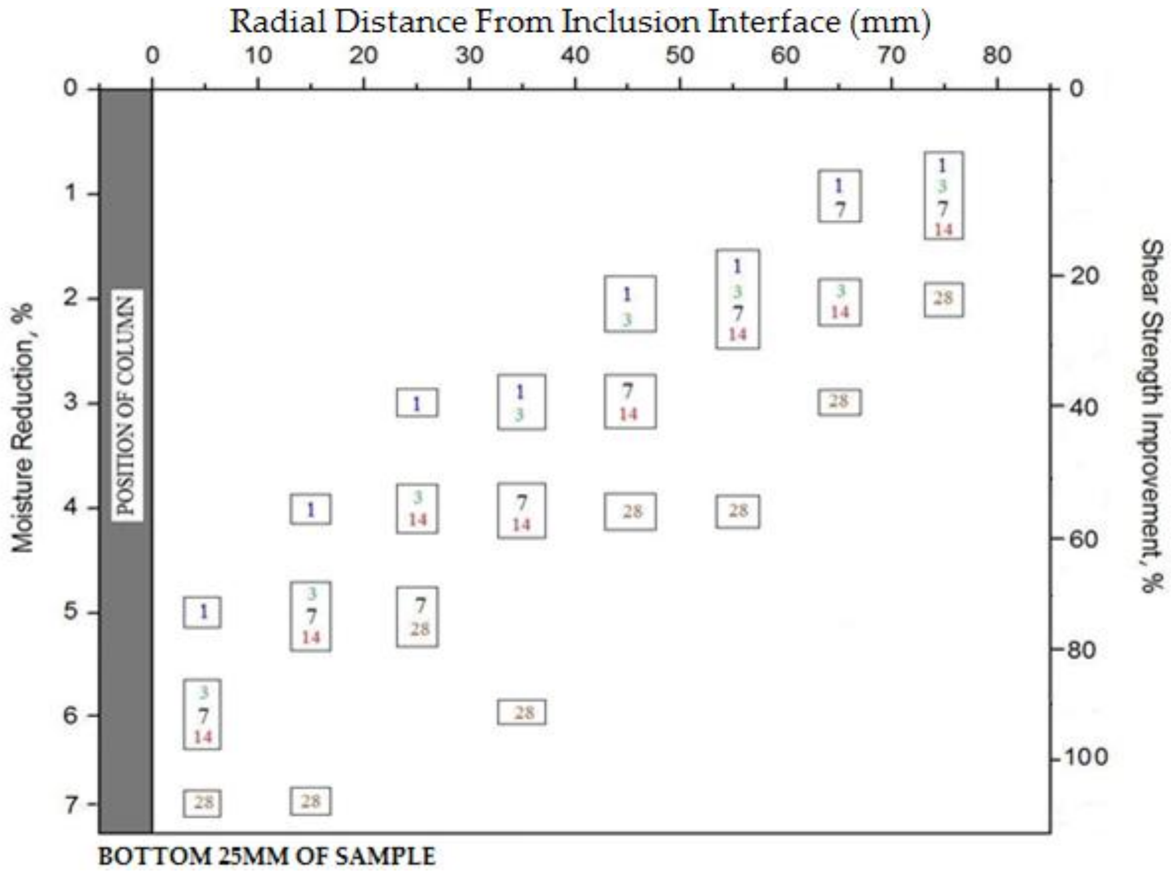


Figure 4.9: contd................

From these results, it is clear to see that the introduction of a 38mm dry cement inclusion has a positive influence on the behaviour of the clay at 40% moisture, particularly in the zone surrounding the soil-inclusion interface[3]; where the moisture content can be seen to reduce in comparison to the untreated clay. The improvement in the zone immediately surrounding the inclusion is the result of the dry inclusion utilising water in the soil, in order to initiate a hydration reaction. As water is absorbed the soil begins to dehydrate and as a result experiences a drop in pore pressure in the vicinity of the inclusion. This drop in pore pressure initiates a pressure gradient, which draws further water from areas of higher pore pressure in order to establish equilibrium. This is the same phenomenon encountered in the clay sample at 60% moisture content, however as the volume of water available in the 40% clay sample is less than the 60% sample, the movement of water through the sample is not as great.

[3] 115% improvement in shear strength recorded at the interface (33% moisture content) in comparison to 12% improvement (39% moisture) at radial distance of 75mm from soil-column interface.

As the volume of water being drawn to the inclusion is not as relentless as in the previous case, the inclusion has the ability to absorb the water when it reaches the interface and as a result no excess water is encountered around the inclusion perimeter and a zone of improvement is maintained.

Similarly to what was anticipated prior to testing, the ability of the cement to cause moisture changes in the surrounding soil reduces with increasing distance from the inclusion; hence so too does the scale of strength improvement. However, with an increase in time, moisture reductions are shown to extend to increasing distances from the interface. This suggests that water is continually being drawn into the inclusion as an improvement in moisture can only result from the inclusions presence in a closed system like this one.

Again the water would initially penetrate the cement through absorption before changing to diffusion once a concentration is set up in the cement: with a difference in moisture between the dry and wet cement providing the concentration gradient. As water enters the inclusion it reacts with the dry cement and initiates a hydration reaction, which causes the cement to harden over time. As the cement hardens the volume of water entering the inclusion continues, but at a reduced rate. The continual ingress of water into the cement is possible, as a result of tensile forces being generated in the capillaries present in the hardened cement matrix. As the microstructural development of the cement continues with increasing contact time with the water, the size of the capillaries in the cement would reduce and cause larger tensile forces to be generated. This continual increase in tensile capillary force would entice more water to be attracted to the inclusion and would explain the moisture changes recorded with increased time.

4.4.2 Hardened Cement Inclusion Testing

Similarly to the samples cured in the 60% kaolin clay, the weight of the inclusion was measured prior to and after testing, in order to provide an indication to the weight of water absorbed by the dry cement. From these results, it was apparent that water was continually being drawn into the sample, but at a reduced rate with increased time. For the first 24 hours after installation the inclusion absorbed approximately 38% water, by weight of dry cement, with 46% and 50% recorded at 3 and 7 days respectively.

The continued absorption of water from the soil, coupled the ability of the pore water to penetrate through the cement to the core of the inclusion was also indicated in the TGA results (Figure 4.10), with the core of the inclusion being seen to progressively hydrate at each stage of curing.

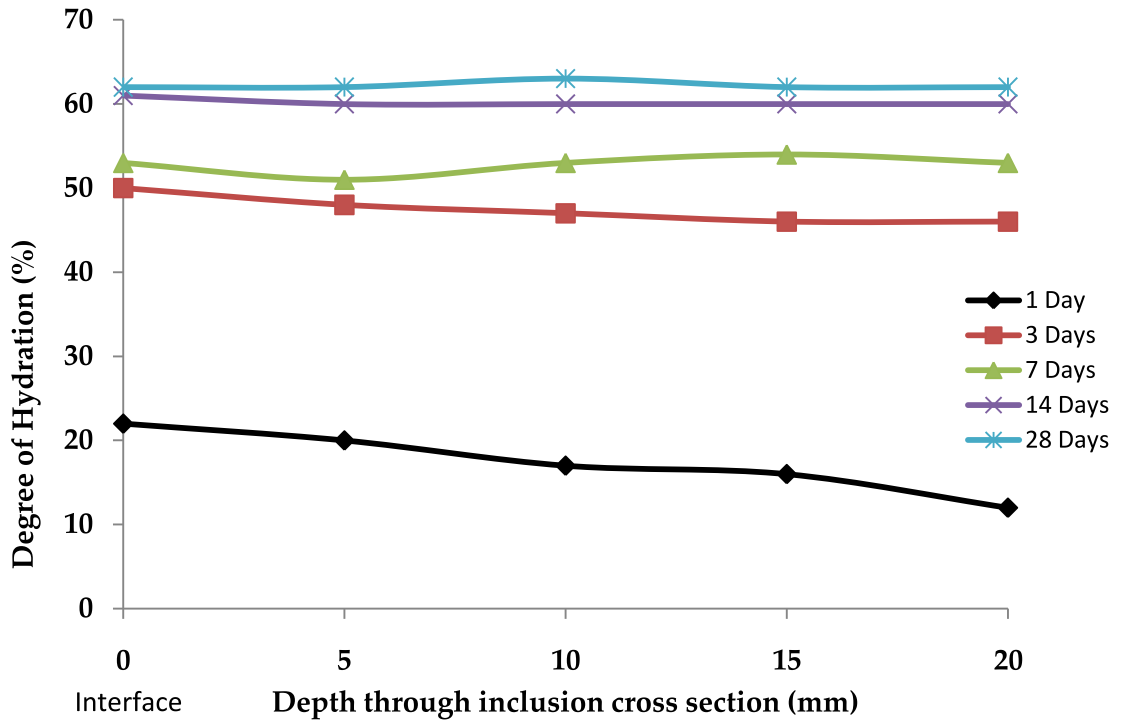


Figure 4.10: Cement hydration throughout 38mm inclusion cross section (40% moisture) with increased contact time

Between 3 to 7 days, the cement at the centre of the inclusion was found to reach the same degree of hydration as the cement at the interface. This indicated that water had successfully managed to penetrate the entire cross section of the inclusion and react with the cement. This was also found to occur in the inclusions cured in kaolin of 60% moisture, however at a much faster rate.

One explanation for this is the reduced volume of water available for hydrating a inclusion in 40% clay soil in comparison to clay of 60% moisture. However, it is more likely that the tensile cracks, observed in the inclusions cured to 60% moisture, played the key role in enhancing the time for water to reach the central core of the inclusion. No tensile cracks were observed in the inclusions cured in 40% clay, therefore the permeability of the inclusion can be said to be far less and as a result water penetration to the central section is retarded. Either way the TGA results indicate that water has the ability to penetrate to the centre of the inclusion even in the absence of tensile cracks.

The weight of water absorbed by the inclusion at 14 days was 53%, with 55% absorbed at 28 days, which showed that continual absorption of water was taking place.

4.4.3 Observations

During testing several differences between clay soils of 40% moisture and 60% moisture were observed, however these mostly centred on the volume of water encountered around the inclusion perimeter and the soil conditions as a result. Quick observations of the hardened inclusion revealed a rough, granular texture on the outer surface of the inclusion, with no visible signs of necking.

One point of interest which did attract the attention of the Author was the formation of a circular yellow band surrounding the entire perimeter of the inclusion (Figure 4.11). These bands were observed in all 40% clay samples after a period of 3 days, with the bands progressing outwards from the inclusion to a distance of 15-20mm from the soil-inclusion interface at 28 days.

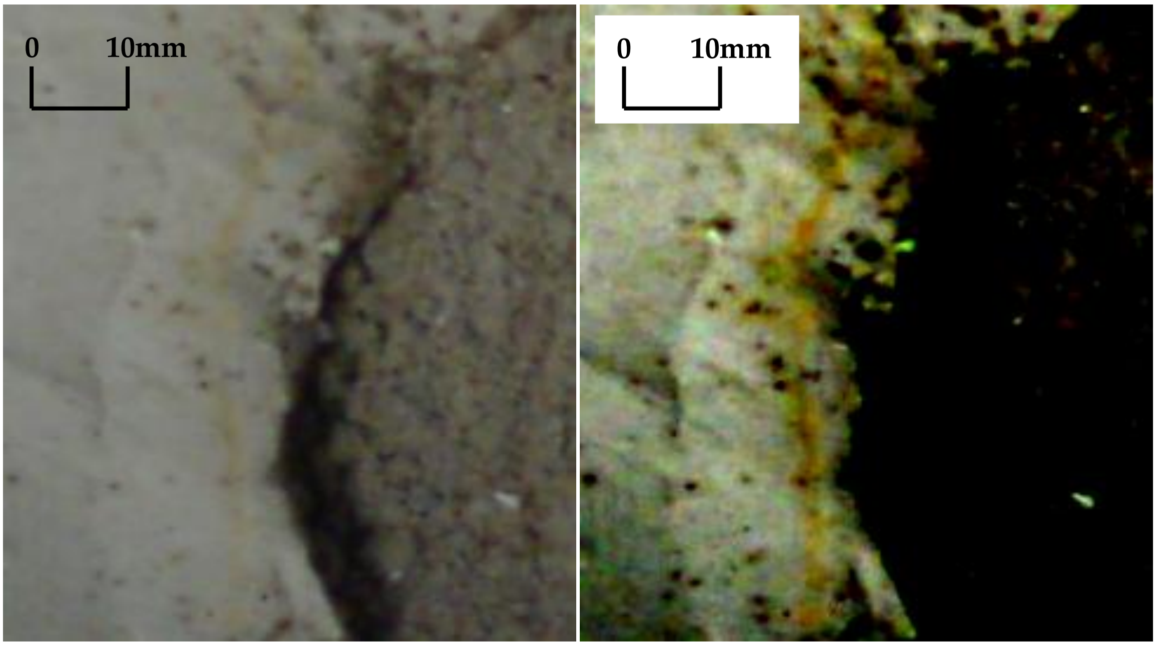


Figure 4.11: Photo showing yellow band around perimeter of the Inclusion (Right image altered in contrast to emphasise line)

It is the believed that the formation of these 'rings' is the result of soluble gypsum (calcium sulfate) migrating out of the inclusion; as the yellow colour of the 'ring' suggests that a sulfate, presumably from the free lime in the Portland cement, is the source. As sulfates are soluble it is likely that the absorption of water by the inclusion would cause the gypsum to be placed in a soluble state. Under a concentration difference the soluble gypsum would then diffuse in the opposite direction to the flow of water into the inclusion, and react with the calcium ions present in the kaolin. This reaction would lead to the formation of ettringite, which has a high affinity for water and strengthens the soil in comparison to the original kaolin. However, the formation of these rings and the presence of ettringite should be further examined and tested, as ettringite could be detrimental to the soil due to its continual expansion on reaction with water. This is out with the scope of this study.

4.4.4 Conclusions Drawn From Analysis of a Single Inclusion at 40% Moisture

It is clear from the results that incorporating a dry cement inclusion in clay soils of 40% moisture has a beneficial influence on the strength characteristics of the soil, as the strength is shown to improve by as much as 115% in comparison to the soil if left untreated. This value was recorded at the soil-inclusion interface where the scale of improvement was shown to be at its maximum, with a reduction in the scale of strength improvement recorded at increasing radial distances from the inclusion.

The contact time between the cement and pore water was shown to have an effect on the strength development of the surrounding soil, with an increase in contact time shown to increase the strength at increasing radial distances. As mentioned, it is the Authors belief that this is the result of increased tensile forces being generated in the capillaries of the hardened cement inclusion, which continually attract water to the inclusion and allow absorption to take place, at an increasingly reduced rate

The TGA results confirm that cement hydration is taking place at the core of the inclusion, with water able to penetrate and maintain a constant supply of water to the core of the inclusion. Continual water absorption was also evident when determining the inclusions weight; with an increase observed over time.

The scale of improvement recorded in clay soil of 40% moisture, using a dry cement inclusion of 38mm diameter, far exceeded the initial expectations. Because of this it was of interest to this project of work, to determine if the strength could be further improved by incorporating dry cement inclusions of larger diameter. The focus of these tests was on the scale and radial distance to which improvement was experienced, as well as the performance of the inclusions themselves, i.e. would water reach and react with the cement at the core of the inclusion.

4.5 70MM INCLUSION IN CLAYS OF 40% MOISTURE CONTENT

4.5.1 Kaolin Clay Soil Testing

The results recorded for the 38mm dry cement inclusions in both 60% and 40% clay moisture samples, led the Author to make the decision that further tests would be carried out solely at 40% initial moisture content. This decision was based on the inability of the cement inclusion to stabilise and improve the properties of the clay soil at 60% moisture at all locations in the sample.

This section of the Chapter will focus on the results recorded for a larger dry cement inclusion of 70mm diameter, with the exact same mixing and testing procedures being performed as for the 38mm inclusion. As mentioned in section 3.2.4, inclusion depths will reach 140mm and radial moisture samples will be extracted to a distance of 160mm from the inclusion interface.

Figure 4.12 shows the effect of introducing a 70mm inclusion with respect to moisture movement in a 40% soil at 1, 3, 7, 14 and 28 days.

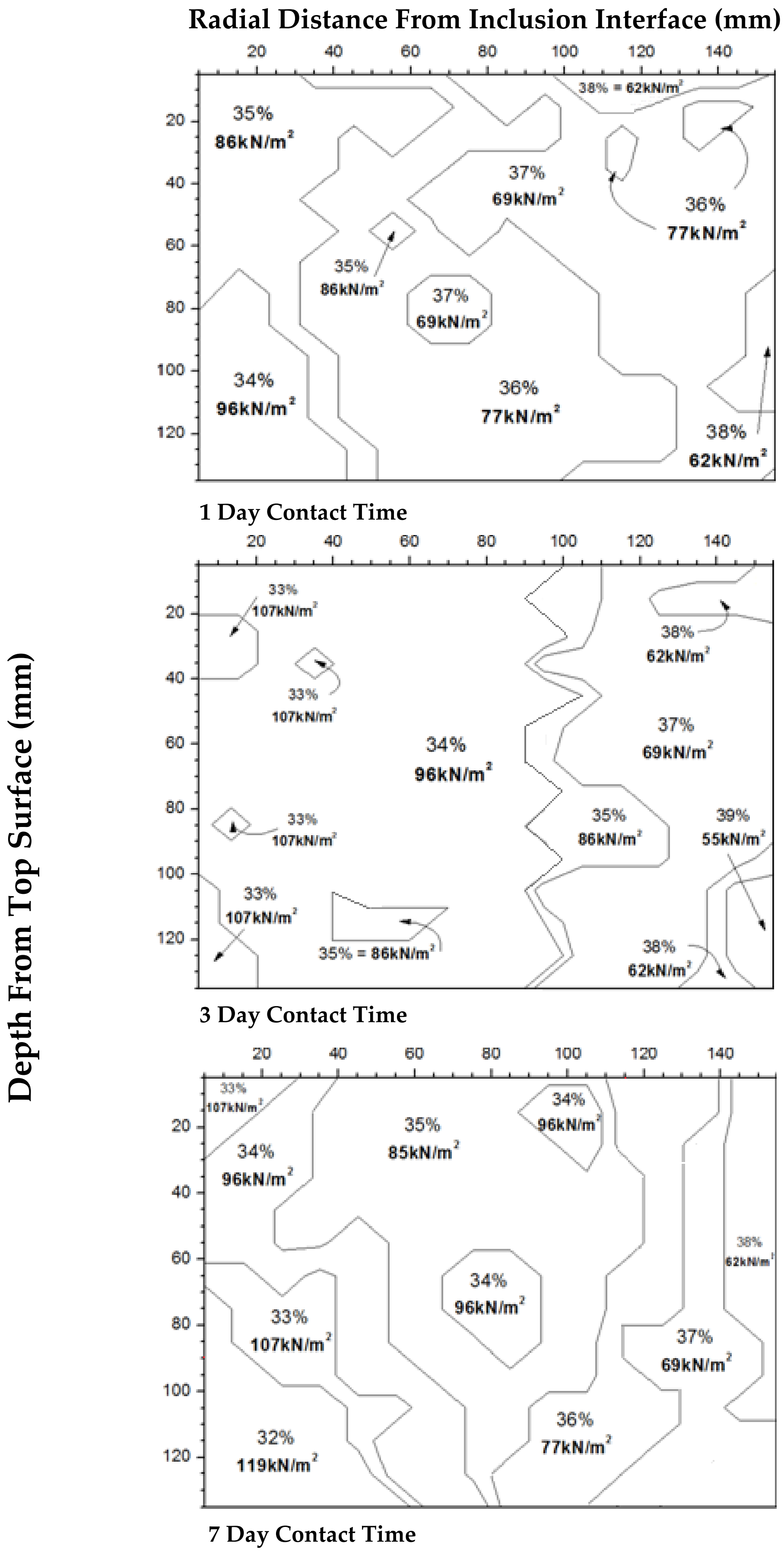


Figure 4.12: Radial moisture movement with increased contact time for 70mm inclusion

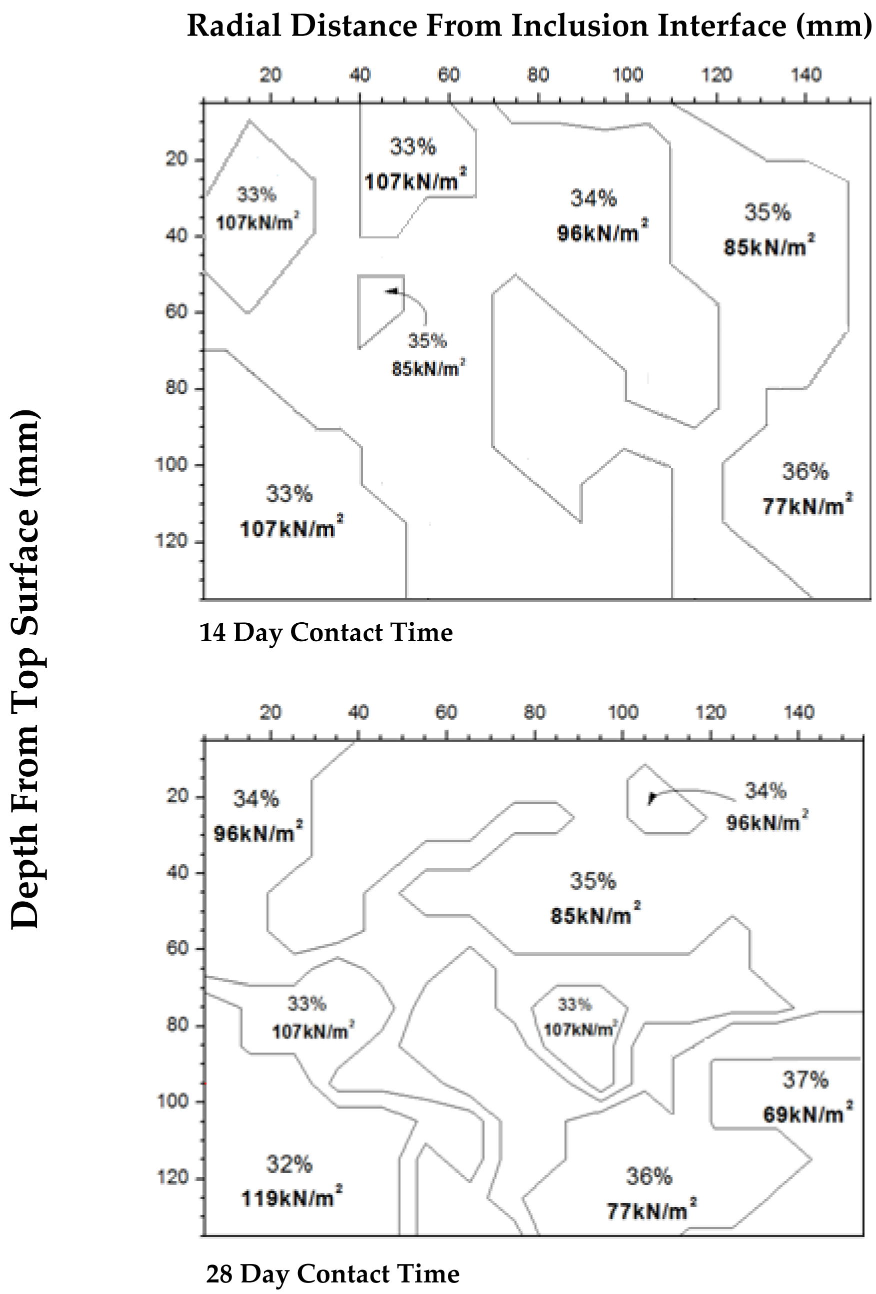


Figure 4.12: contd.......

Summary of Graphs

As the length of these inclusions are 140mm, the decision was taken to split the inclusion into three 45mm sections along its length and take the average moisture content at every 10mm interval for each of the three section. This allowed the summary graphs in Figure 4.13 to be produced, where the moisture content was considered for each length of time the cement remained in contact with the pore water.

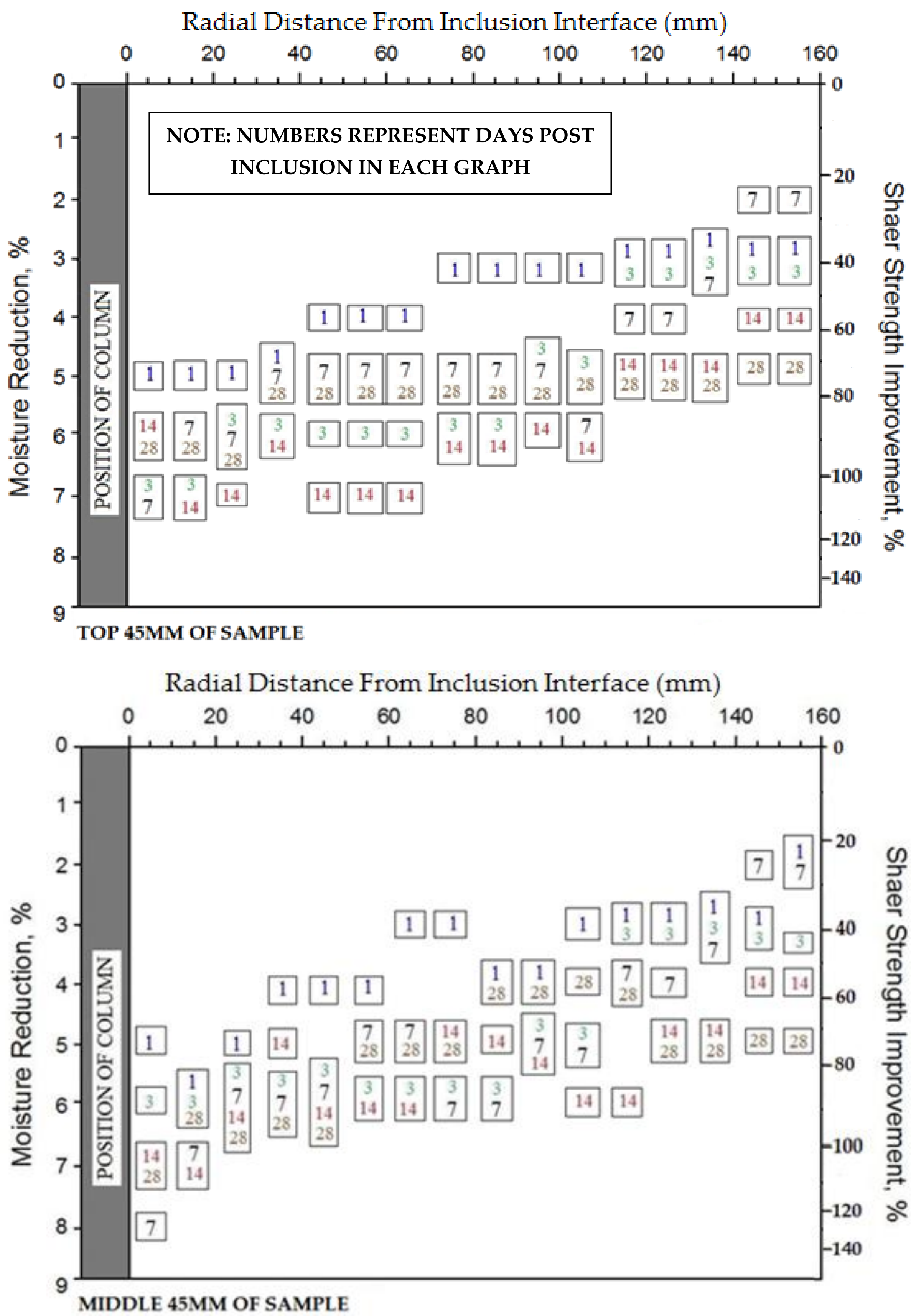


Figure 4.13: Summary of the changes in moisture content experienced over time, with a 70mm inclusion in a clay soil of 40% moisture

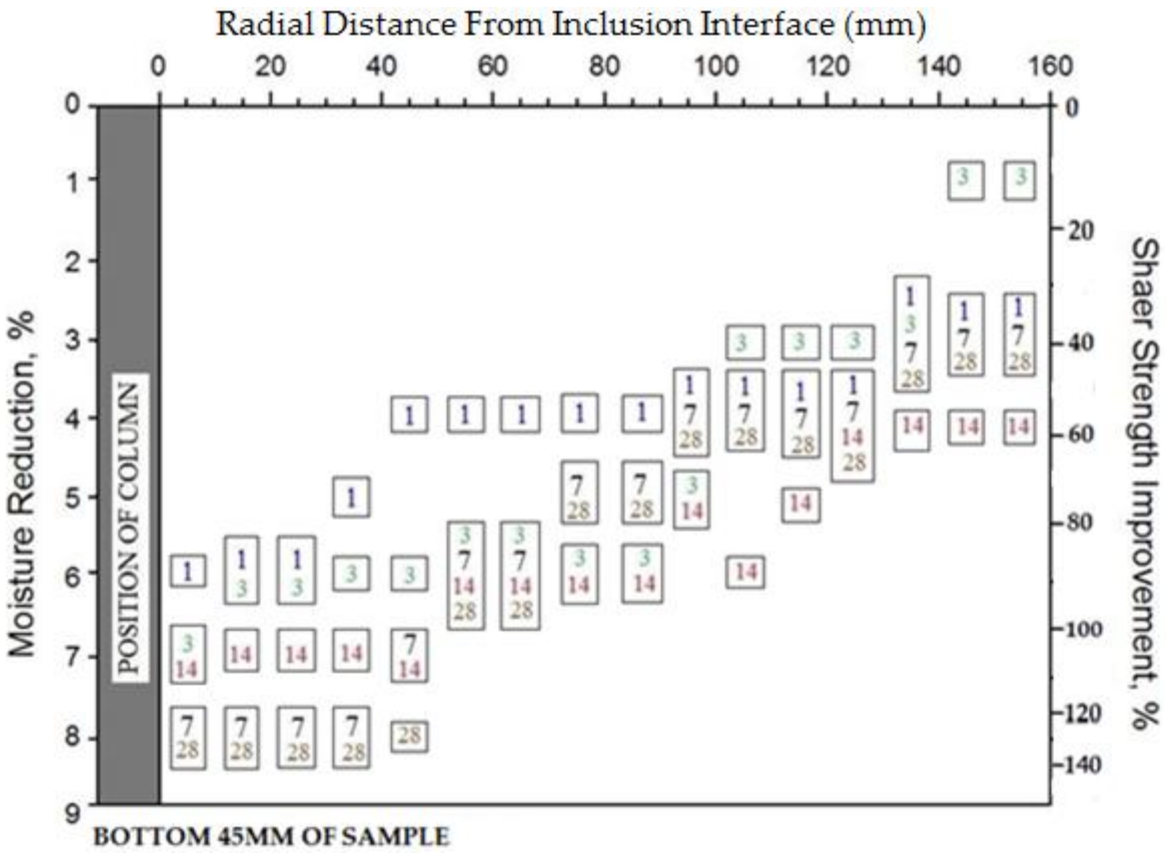


Figure 4.13: contd……….

From these results it can be said that an increase in inclusion diameter has a significant influence on the radial improvement in a clay soil of 40% moisture. Improvement is recorded for the full length of the 160mm where moisture extractions were taken from the clay, in comparison to the 38mm inclusion where improvement was limited to approximately 80mm from the interface. Again the largest improvement was recorded at the interface with the level of moisture change shown to reduce with an increase in radial distance. A shear strength increase of 139% (32% moisture; 119kN/m^2) was recorded at the interface for the 7 and 28 day test samples, in comparison to the control sample where the soil is in an untreated condition.

The extended radial influence of the 70mm inclusions is the result of a greater volume of cement being necessary to fill the cavity hole and form the inclusion. By increasing the cement content, the volume of pore water required to satisfy the cements natural affinity for water is increased. Therefore, a greater degree of absorption is achieved and a larger pressure gradient is induced in the soil. This increases the radial influence of the inclusion, which is reflected in the results.

Similar to the 38mm inclusion, the radial influence of the 70mm inclusion is shown to develop with an increase in time, with a continual reduction in moisture observed at increasing radial distances. Again this is most likely the result of tensile forces from the capillaries in the inclusion removing water from the soil, causing a pressure gradient to effectively remain in place. The performance of hand shear vanes provided further evidence of a dewatering effect occurring at increased curing periods and at increasing radial distances during these periods.

4.5.2 Hardened Cement Inclusion Testing

TGA was again performed at 5mm intervals throughout the inclusion; from the interface to the central core of the inclusion. The results can be viewed in Figure 4.14.

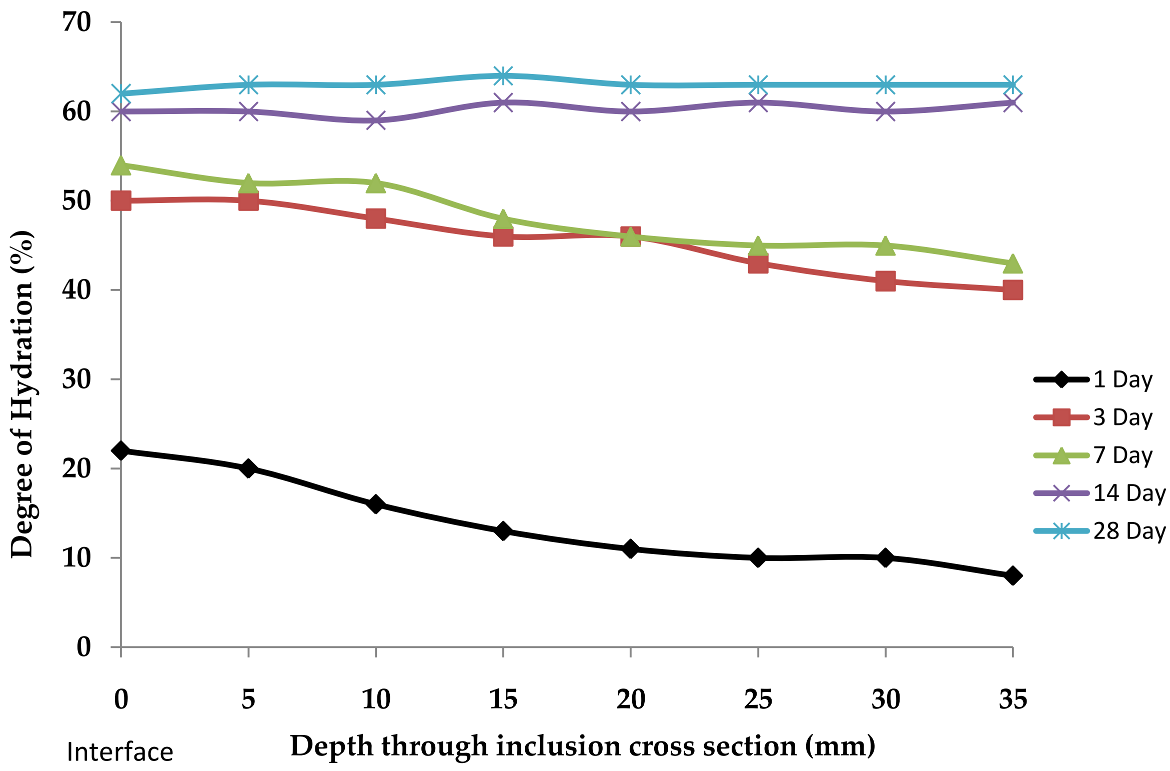


Figure 4.14: Cement hydration throughout 70mm inclusion cross section

From these results it is clear to see that the degree of hydration at the centre of the 70mm inclusion takes 14 days to match the value at the interface. This is a longer process; in comparison to the 38mm inclusion and is to be expected as a larger inclusion increases the distance, and hence time, for water to penetrate to and react with the cement at the centre of the inclusion. Once water has reached the central section of the inclusion it continues to hydrate at the same rate as the interface, indicating that water has a sustained ability to penetrate the full length of the inclusion and react with the cement.

Again an increase in the weight of water absorbed was recorded with time. This supports the results from the TGA analysis. The absorption of water in the first 24 hour period was recorded as 32%, by weigh of dry cement. However, an interesting point to note about the 70mm inclusion at 1 day was that the centre of the inclusion appeared to remain completely dry and unhydrated, as there was no change in colour or hardening of the cement, as was the case with all other inclusions up to that point. In fact, the cement was so loose and easily displaced that the Author was able to carve a hole in the centre of the inclusion (as shown in Figure 4.15) by simply moving his thumb over the area in a circular motion.

Figure 4.15: Unhydrated cement removed from core of 70mm inclusion at 1 day

The hole was approximately 30mm in diameter with 20mm hardened cement on either side. TGA results suggest that a slight degree of hydration has taken place in this region of the hole however no interaction with water seemed evident to the Author. Every sample cured for periods greater than 24 hours showed a hardened, solid mass at all locations in the inclusion. Again this would be the result of water continually penetrating through the inclusion causing the cement to hydrate. The inclusion was shown to absorb approximately 59% water by the end of the 28 day tests, which surpasses the weight of water absorbed in the 38 mm inclusion. Again this was anticipated before proceedings as increasing the cement content would lead to a higher demand for water.

4.5.3 Observations

After extracting the hardened cement inclusions from the kaolin clay samples, it was apparent that an onion skin effect throughout the entire cross section of the inclusion had occurred (Figures 4.16 & 4.17). This effect was apparent from two separate observations. Firstly the surface texture was again very rough, similar to the 38mm inclusions, however was easily removed to reveal underlying layers. Careful removal of several very thin layers was performed by the Author by simply rubbing a layer until it crumbled away from the sample. Each layer revealed a different texture, which is evident in Figure 4.16 and 4.17.

Figure 4.16: 70mm hardened cement at 7 days

Figure 4.17: 'Onion skin' effect throughout hardened cement inclusion

This layered effect was also apparent within the colour profile of the hardened cement throughout its cross section. The interface was a far darker shade of grey than the central section of the inclusion at all stages of curing up to 28 days (Figure 4.18). This indicates that the cement at the interface has a greater interaction with the water than in the centre of the inclusion. However, with an increase in time TGA results show that the degree of hydration at the core of the inclusions steadily increases towards the same level of hydration recorded at the interface. Therefore, water must be continually drawn into the centre of the inclusion with time, in order for this to occur.

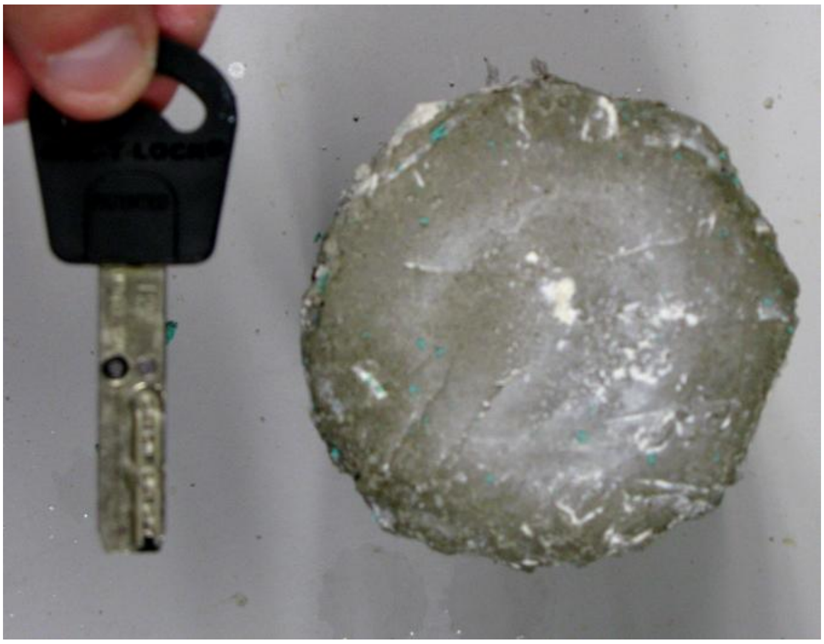


Figure 4.18: Colour profile in 70mm cement inclusion (key shown for scaling purposes)

This 'onion skin' effect and the increased degree of hydration at the core of the cement, is most likely the result of meniscus forces being set up within the samples. These meniscus forces would cause a chain-like reaction between unhydrated cement and the pore water to continue throughout the inclusion cross section.

In simpler terms a layer of cement reacts with the pore water to hydrate. Once a certain degree of hydration has occurred, a capillary is formed leading to a tensile force which draws more water into the sample. This water then reacts with an underlying layer of unhydrated cement, which again hardens and causes a tensile force in the capillary, and so forth (Figures 4.19 and 4.20). This process would explain the continuous hydration of the central section of the cement inclusion even after the interface has sufficiently hardened.

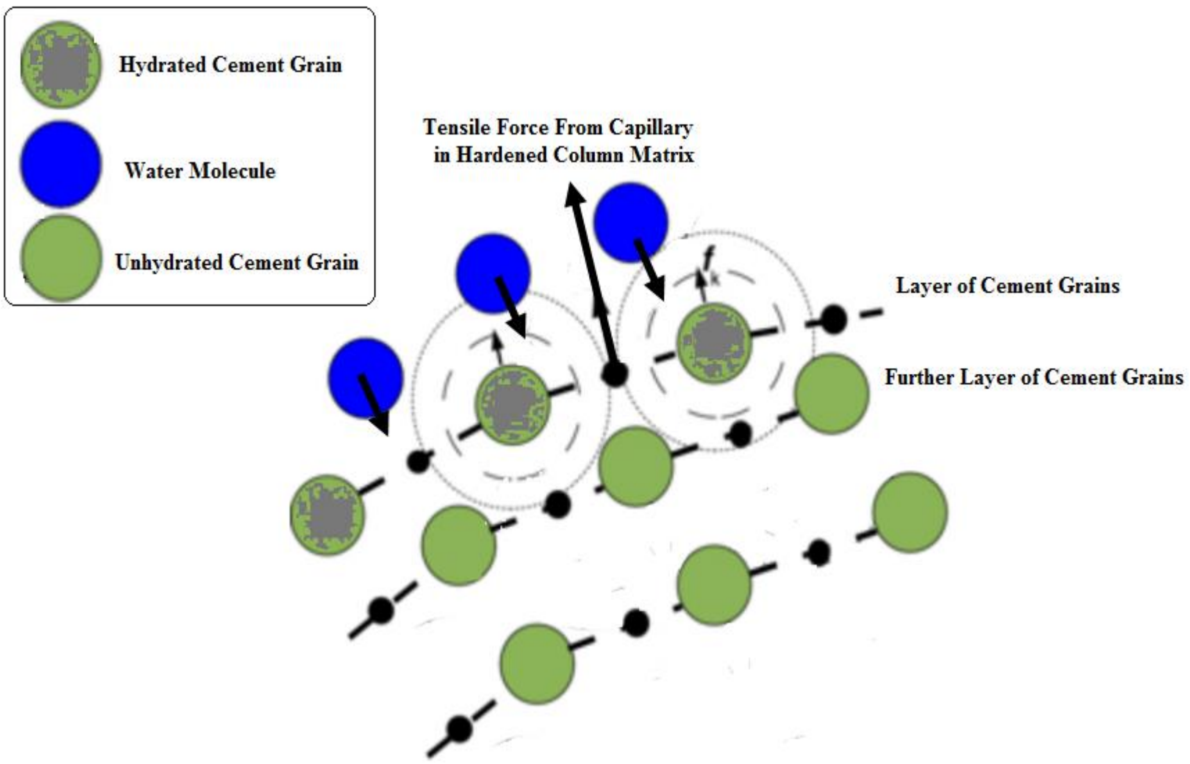


Figure 4.19: Meniscus force drawing water to centre of inclusion

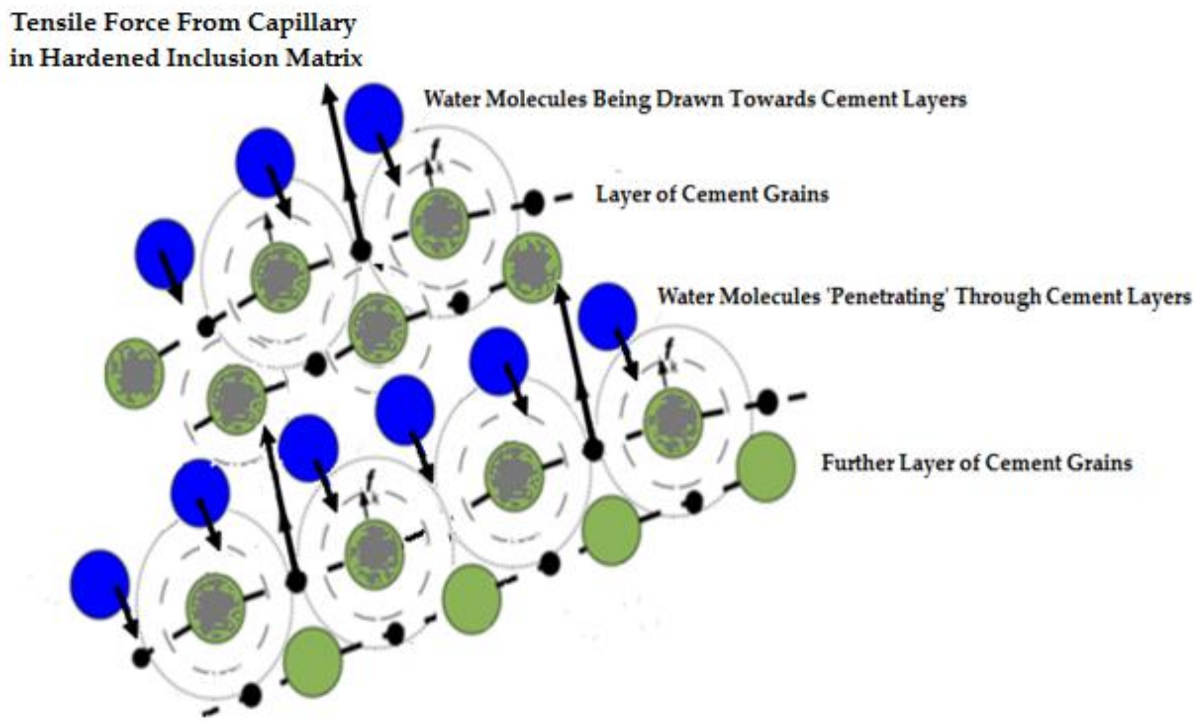


Figure 4.20: Continual absorption of water to centre of inclusion

Similarly to the 38mm inclusion, no tensile cracks were visible in any of the 70mm inclusions stored in 40% moisture content. This seems to be a consequence of the vast volume of water available in the 60% clay sample as this causes rapid hydration and the formation of cracks. There were also no visible signs of necking along the inclusion length.

Again yellow bands were visible around the perimeter of all inclusions after 3 days curing, and once again these extended to a radial distance of approximately 15-20mm from the inclusion interface. Therefore, it does not seem that an increase in the cement content has any influence on the migration of sulfates out of the inclusion. Again this is out with the scope of this current project of work.

4.6 100MM INCLUSION IN CLAY OF 40% MOISTURE CONTENT

An investigation involving a cement inclusion of 100mm diameter was conducted solely at 28 days. This decision was taken based on the previous tests results of the 38mm and 70mm inclusions, which provided evidence that the core of the inclusion continued to have access to water until hydrated to the same degree as the interface. This was achieved at some stage between 3 and 7 days for the 38mm inclusion and 14 days for the 70mm inclusion. This suggested that longer times were necessary for larger inclusions. As all tests on 38 and 70mm inclusions continued to a period of 28 days, it was decided that this would be a suitable length of time for the 100mm inclusion and would allow comparisons to be drawn between inclusion diameters.

4.6.1 Observations

At 28 days, visual observations of the hardened inclusion and soil were conducted by the Author prior to extracting moisture samples. The surface of the soil displayed clear indications that improvement had taken place; with the Authors thumb unable to penetrate through the soil surface. Instead only small indentations were produced suggesting the soil was considerably stiffer in comparison to the control[4]. This was continually experienced up to a radial distance of approximately 125mm, at which point larger indentations with the thumb became possible. This provided an indication that the

[4] The control virgin soil at 40% moisture content facilitated complete penetration of the authors thumb. Therefore, any increase in the soil stiffness would oppose the penetration of the author’s thumb and increased effort would be necessary.

extent of improvement experienced by the soil was reducing with increased radial distance, similar to that experienced with both the 38mm and 70mm inclusions.

Signs of a dewatering effect in the clay were also visible in the form of slight settlement being noted around the perimeter of the inclusion. This settlement was very small, with an estimated settlement of approximately 2mm from the surface being apparent up to a radial distance of 13mm from the soil-inclusion interface. Settlement had not been experienced in any previous experiments involving the dry cement inclusions at smaller diameter. It is believed that settlement was visible in this instance, due to the high level of cement placed in the inclusion. Again this would lead to a larger volume of water being absorbed by the inclusion, with an increase in the potential to cause settlement.

Initial observations also identified a large tensile crack, of length 58mm in the centre of the inclusion (Figure 4.21). Similarly to the inclusions cured in clays mixed to a moisture content of 60%, this crack is more than likely the result of volume changes during cement hydration. As shrinkage is known to increase with increased cement content, this could explain why tensile cracks are not observed for smaller inclusion cured in 40% moisture conditions.

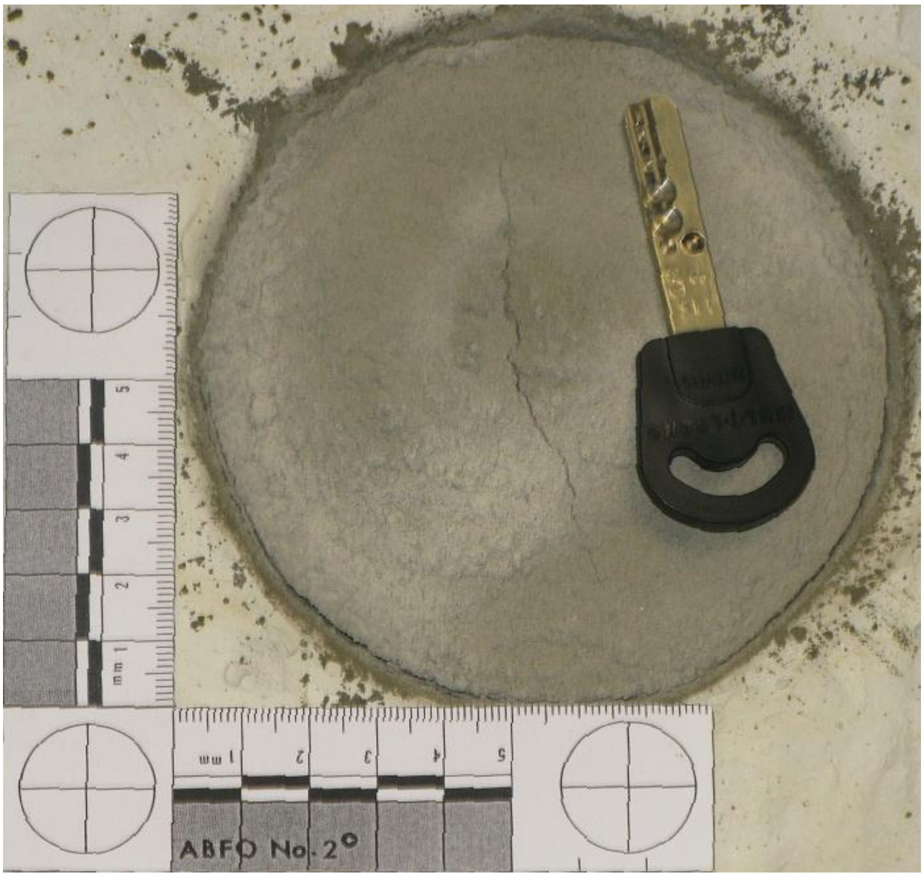


Figure 4.21: Tensile crack through centre of 100mm dry cement inclusion

Further evidence of shrinkage was provided in the form of separation between the hardened inclusion and the soil interface, which was visible at all locations around the inclusions perimeter (Figures 4.22 and 4.23). It is the Authors belief that this separation is the result of plastic shrinkage and was measured to be in the region of 2-3mm at some points. Again this would be the result of a greater volume of dry cement being necessary to form the 100mm inclusion, with an absence of aggregate contributing to the level of shrinkage; as aggregates provide physical resistance to limit shrinkage. Autogenous and drying shrinkage are also likely to have occurred, however these are out with the scope of this investigation and further studies will have to be undertaken to fully understand the causes of shrinkage and its influence on the level of improvement provided to the surrounding soil.

Figure 4.22: Plan view of 100mm inclusion

Figure 4.23: Exploded view of separation between inclusion and clay

It is evident from Figure 4.22, that the centre of the inclusion displays a different shade of grey in comparison to the interface. During initial observations, it was feared that penetration of water had not been successfully achieved through to the core of the inclusion, as the cement looked dry and loose. This was not the case. The texture of the hardened inclusion was the same as all other inclusions cured in 40% moisture, with a rough, granular consistency being recorded.

Again a yellow ring was observed around the perimeter of the inclusion, the radial distance of which was the same as for the previous inclusions. Upon extracting the hardened inclusion there were no signs of necking along its length, and the same 'onion skin' effect was apparent throughout the samples cross section. This strengthens the belief that meniscus forces are causing the cement to hydrate through to the central section of the inclusion (as detailed in Figures 4.19 and4.20).

4.6.2 Kaolin Clay Soil Testing

As mentioned moisture extractions were only taken at 28 days from the kaolin clay sample stabilised using a 100mm dry cement inclusion. Again, these results were plotted in a contour graph (Figure 4.24) in order to illustrate the movement of the moisture as a result of the dry cement inclusion utilising pore water in order to hydrate.

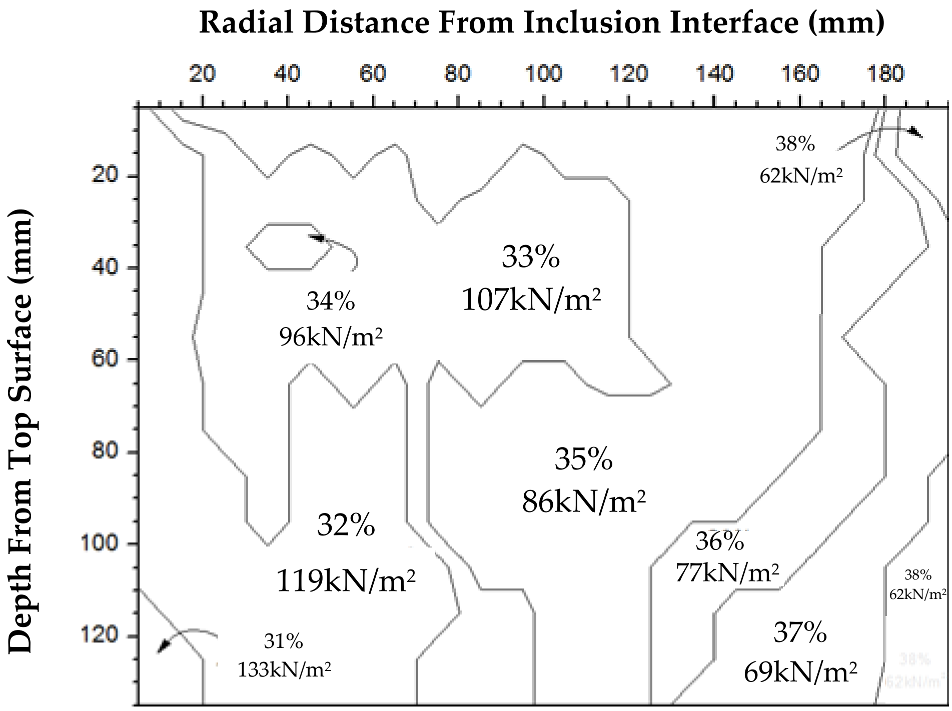


Figure 4.24: Radial moisture movement for 100mm cement inclusion at 28 days

The pattern of moisture movement is similar for this inclusion as in the two previous cases, with the most significant reduction in moisture recorded at the interface (167% improvement in shear in comparison to an untreated sample, indicated by the 31% moisture reading in Figure 4.24).

Again, the results indicate that an increase in the diameter of the inclusions can have a positive effect on the soil at increasing radial distances from the inclusion interface, with improvement recorded for the full 200mm length away from the inclusion to which samples are extracted. This reiterates the fact that increasing the cement content in the inclusion allows a larger volume of water to be absorbed by the cement and a larger pressure gradient to be experienced in the soil. This is known to be true as the soils desire to equilibrate causes moisture to migrate towards the inclusion. If this movement is because of a larger pressure difference then the volume of water required to achieve equilibrium increases, hence a larger radial influence is the result.

4.6.3 Hardened Cement Inclusion Testing

Upon successful extraction of the inclusion, it was discovered that approximately 48% water, by weight of dry cement, was absorbed at 28 days. This was only a slight reduction in comparison to the 38mm and 70mm inclusions at the same stage of curing. However, TGA results indicate that the centre of the 100mm inclusion has hydrated to a lower degree of hydration compared to the interface (Figure 4.25).

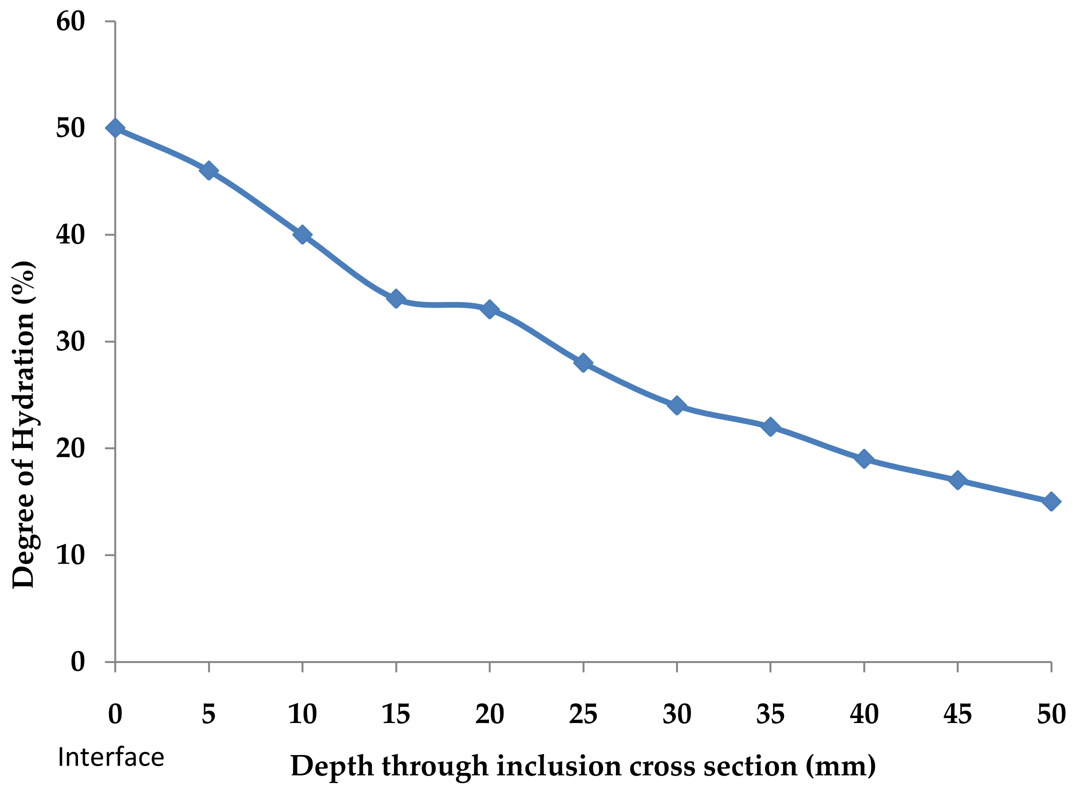
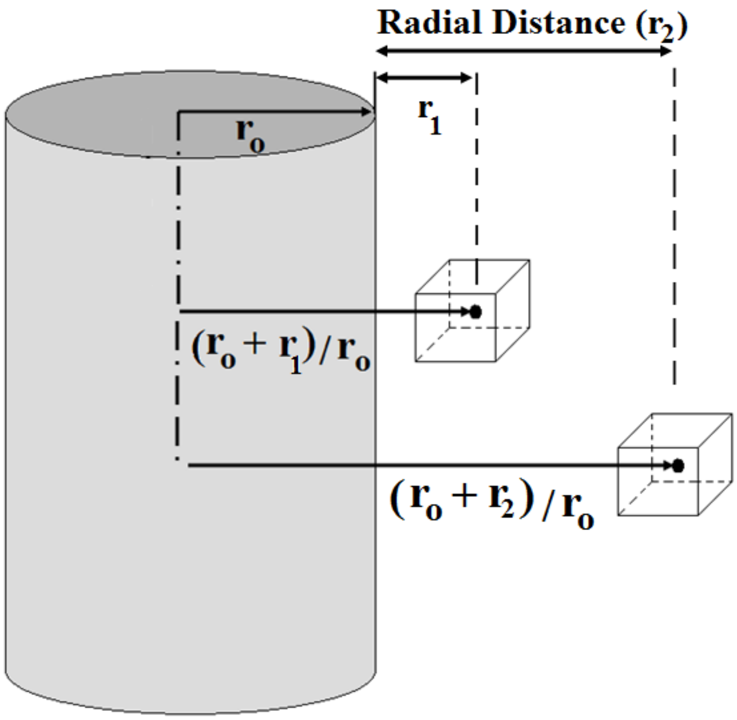


Figure 4.25: Cement hydration throughout 100mm inclusion cross section at 28 days

This would suggest that longer times are necessary for larger inclusions, as although water is able to penetrate to the cement at the core of the inclusion, its hydration reaction is not as well developed as at the interface. It could also potentially be the case that inclusions larger than 70mm never achieve the same degree of hydration at the core as at the interface. However, it is difficult to justify these statements without further research into larger scaled inclusions being conducted.

4.6.4 Conclusions Drawn From Increased Inclusion Diameter on Radial Improvement of Clay Soil at 40% Moisture

From the results obtained in the 70mm and 100mm inclusion experiments, it is fair to say that increasing the inclusion diameter has a significant impact on the radial distance to which the soil experiences dehydration; as by enhancing the inclusion diameter reduced moisture contents are recorded at increasing radial distances from the inclusion interface. However, in order for a direct comparison between the inclusion sizes to be performed it is necessary to normalise the data, i.e. compare radial dehydration influence with respect to inclusion diameter (Figure 4.26). As the 100mm cement inclusion was only tested at 28 days, a comparison between the moisture changes across the three inclusion diameters was only possible at this time.



Where:

r_o = Radius of inclusion

r_1 = Radial distance to centre of first moisture sample (mm)

r_2 = Radial distance to centre of second moisture sample (mm)

r_3 = etc, etc

Figure 4.26: Normalised data for a comparison between inclusion diameters at 28 days

By taking the average moisture content along the length of the inclusion at normalised locations in the sample (in this case the only common normalised locations between 38, 70 and 100mm inclusion diameter were 2.3, 3.6 and 4.9), Figure 4.27 can be produced which best depicts the influence of the inclusion diameter on the radial dehydration of the surrounding soil.

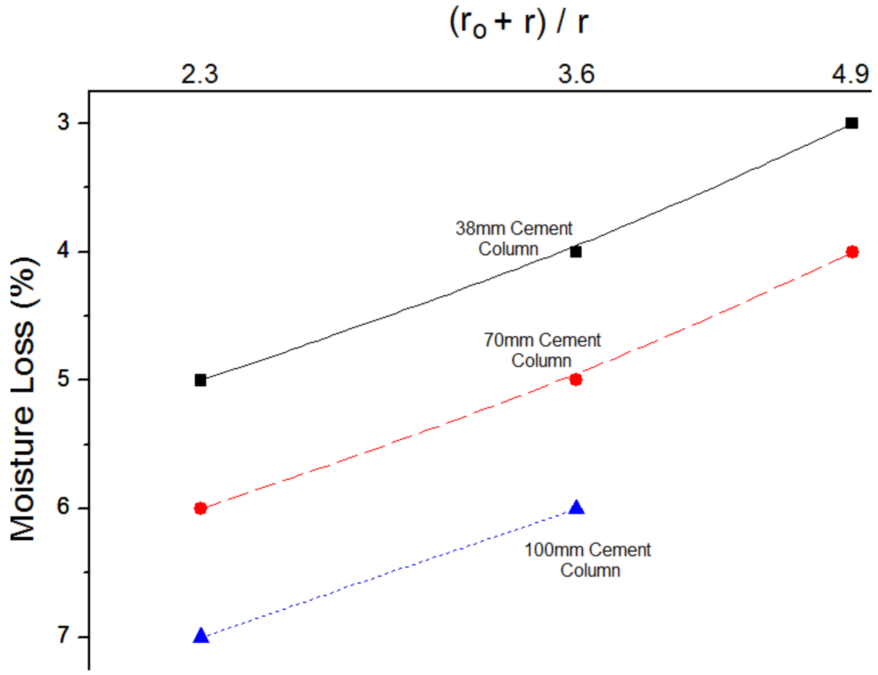


Figure 4.27: Normalised comparison of inclusion diameters, with respect to radial influence at 28 days

From these normalised results it is fair to say that increasing the inclusion diameter has a beneficial impact on the radial influence to which the soil experiences dehydration. By enhancing the inclusion diameter, improvements in shear strengths were recorded at increasing radial distance from the interface with longer an increase in time shown to benefit the soil. Again, this has been accredited to the increased cement content, necessary to form the inclusions, drawing in a larger volume of water by weight of dry cement. This increase in absorbed water causes a larger pressure gradient to be provoked in the soil. The larger the pressure gradient, the larger the volume of water required to migrate towards the inclusion in order to establish equilibrium in accordance with the Principles of effective stress. Thereby, a greater radial influence is experienced.

It could not be confirmed within this project of work, whether or not there is a limitation to the inclusion diameter at which point water ingress into the central core of the inclusion is not permitted. The TGA results from the 100mm inclusion indicate that the central section of the inclusion has been granted access to water as a hydration reaction has been seen to occur, however not to the same degree as the interface. This could indicate that inclusions of 100mm do not permit a constant ingress of water through its entire cross section; however it is believed that a longer curing time is simply required. Further investigatory work is suggested involving larger column diameters.

4.6.5 Filter Paper Suction Test Results

The filter paper suction test is an 'indirect method' of measuring soil suction and provides an indication of the clay soils ability to attract and retain pore water. The test works under the assumption that an initially dry filter paper will equilibrate, in terms of water vapour flow for soils with a specific suction using the equation:

$$\log \text{suction (kPa)} = 5.327 \times 0.0779 w_t$$

The filter paper suction tests were performed on hand mixed samples prepared to 40% moisture, as detailed in section 3.5.3. The first set of tests involved three 100mm clay control samples, with filter tests performed on each sample in the absence of a dry cement inclusion. This allowed the general range of suction forces generated in a 40% kaolin clay sample to be determined (Table 4.2). An average value was then taken as the control value.

Table 4.2: Range of suction forces in clay of a moisture content of 40%

Specimen Description	Filter Paper Moisture Content, %	Measured Matrix Suction, kPa	Total Suction*, kPa
Control Sample 1	40.2	157	181
Control Sample 2	39.9	165	179
Control Sample 3	40.4	151	190

*Total suction measured by the filter paper suspended above the clay sample.

The second set of tests were performed in order to examine the effects of introducing an inclusion of dry cement on the clay soils suction values. It was expected that an increase in suction would result from a loss of moisture in the clay as a result of the cement utilising pore water. For the purposes of this set of tests two clay samples, again of 100mm diameter, were extracted from the same sample. A point of interest was whether or not the radial dewatering influence of the inclusion; which was apparent in the single inclusion analysis of section 4.3, would also be reflected in the soil suction values at increasing radial distances from the inclusion. The results can be viewed in Table 4.3.

Table 4.3: Matrix suction values recorded from the Filter Paper Suction Test

Position of Filter Paper	Filter Paper Test 1		Filter Paper Test 2	
	Moisture Content, %	Matrix Suction, kPa	Moisture Content, %	Matrix Suction, kPa
Soil-inclusion interface	34.2	460	33.4	531
10mm from interface	35.6	358	32.7	602
20mm from interface	33.0	570	33.0	570
30mm from interface	34.1	468	32.1	669

The results recorded from the filter papers at increasing radial distances from the soil-inclusion interface did not provide the data which was anticipated prior to proceedings. It had been hoped that the suction recorded at the interface would be far greater than the suction recorded at increasing radial distances from the inclusion interface. This was expected as the dry cement would utilise the pore water in the clay soil in a zone surrounding the inclusion, causing an immediate reduction in moisture within this zone to occur. This reduction in moisture would lead to an increase in soil suction and a pressure gradient to be set up in the soils matrix.

It was hoped that this pressure gradient would cause water to migrate towards the inclusion, causing further reduction in moisture at increasing radial distances from the inclusion interface to be noted. As the inclusion has been shown to continually absorb water, the increase in suction experienced at increasing radial distances would not be to the same degree as at the interface. This is unfortunately not reflected in the results.

Instead a more random process appears to have occurred as a result of introducing the dry cement. Indeed an increase in suction is recorded in the clay sample with the presence of a dry cement inclusion; however these appear to follow no particular pattern. A greater value of suction can be seen at a radial distance of 20mm than at the interface, with a suction value of 570kPa which was not expected. Similarly a suction value of 468kPa is recorded a radial distance of 30mm, which is slightly higher than the interface with 460kPa.

Conclusions Drawn From Filter Paper Suction Test

The filter papers in contact with the kaolin clay do not exhibit consistent results, with a wide range of suction values found at increasing radial distances from the inclusion interface. This could due to a number of factors, such as poor contact between the filter paper and the clay sample. However, this test was performed in the exact same manner as the control samples, which although varied in suction values were fairly consistent with 157 ± 8kPa. The Author also ensured that the moisture of the filter papers was performed immediately after each one was removed from contact with the soil, so drying of the filter paper is not likely to have influenced the results.

It is possible however, that the continuous absorption of pore water by the dry cement for the duration of the test could have influenced the filter papers ability to gain equilibrium within the soil. This would have influenced the results, however without further analysis this cannot be confirmed.

This test was performed to further understand the moisture movement and dewatering process involved in a soil, by examining the change in soil suction values as a result of incorporating a dry cement inclusion. Unfortunately, this could not be achieved with the results obtained and further investigation using this technique is proposed.

4.7 CALCIUM SULFOALUMINATE CEMENT INCLUSION (CSA)

For this section of the Chapter, an investigation was conducted into whether or not replacing Portland cement (PC) with Calcium Sulfoaluminate (CSA) cement would provide greater strength improvements to the surrounding soil. As discussed in Chapter 3, CSA is known to have environmental benefits in comparison to PC and has the potential to provide enhanced strength based on the findings of Åhnberg (1995); who concluded that fast setting cement produces higher shear strengths than traditional Portland cement; when used to form lime-cement columns. The CSA inclusion was formed to a diameter of 38mm, with moisture results determined 24 hours after installation (Figure 4.28).

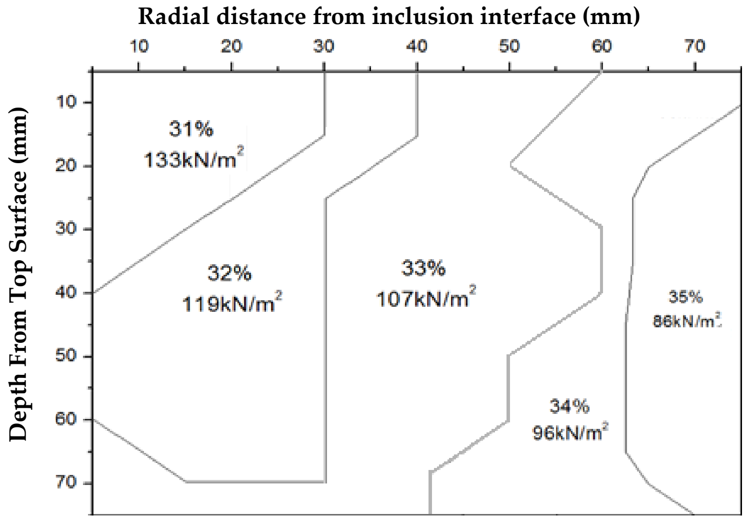


Figure 4.28: Radial moisture movement for 38mm CSA cement inclusion at 1 day

Moisture content samples were intentionally taken 24 hours after installation in order to allow a comparison between the CSA cement and the 1 day results for a 38mm PC inclusion to be conducted (Figure 4.29).

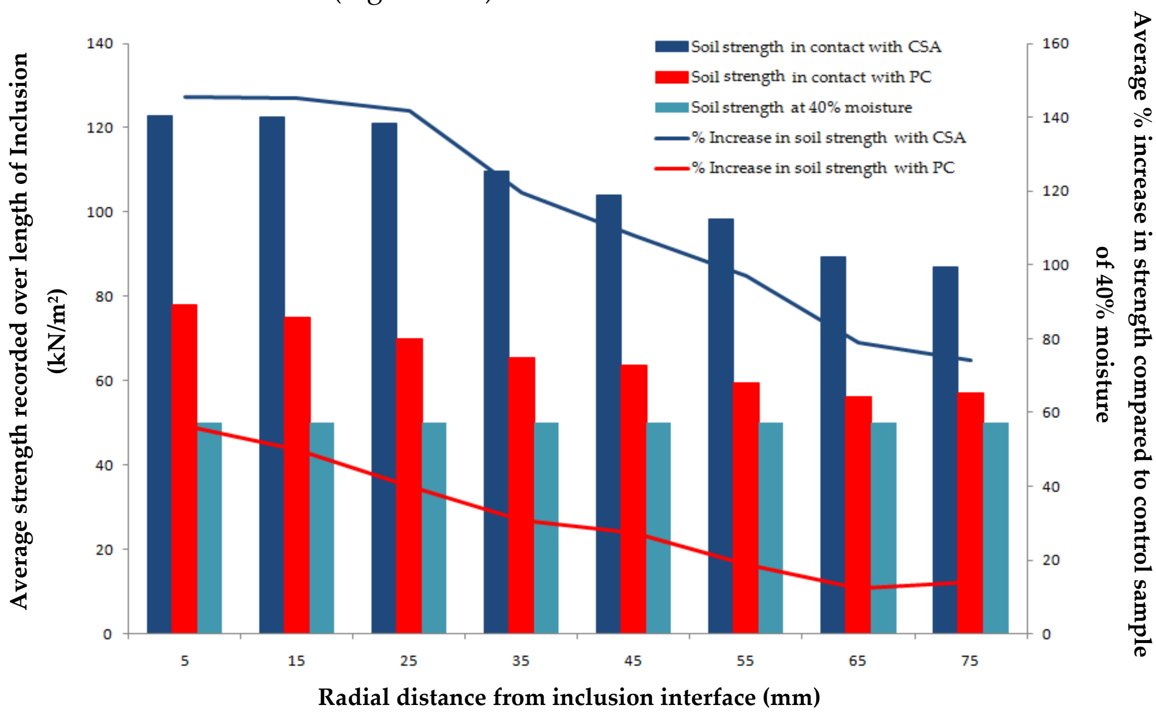


Figure 4.29: Comparison between CSA and PC in ability to strengthen surrounding soil

From these results, it can be concluded that replacing PC with CSA has a beneficial influence on the surrounding clay soil during the initial 24 hour period after installation. It was observed that an increase in soil strength of 146% was achieved at a 5mm distance from the CSA inclusion in comparison to the 56% recorded for the soil in contact with a PC inclusion. It is likely that rapidly dewatering the soil, as is the case with CSA, creates an influence similar to increasing the cement content of the inclusion, with a much larger pressure gradient created in the soil. It therefore seems advantageous to incorporate a quick setting material into the inclusion in place of PC. However, further use of this material is not performed in this project of work. Further investigatory work is recommended by the Author as the physical results and economical benefits of incorporating CSA are clearly more advantageous to the system than simply using PC alone.

4.8 CHAPTER SUMMARY

From the results and observations recorded in this section of work the following conclusions can be made:

- Incorporating a 38mm dry cement inclusion into a clay soil of 60% moisture, has been shown to have two effects on the strength properties of the soil. The cement inclusion was shown to weaken the strength in the region immediately surrounding the inclusion; as a result of moisture build up. Whilst an increase in strength was experienced at increasing radial distances from the inclusion interface; as a result of water migrating towards the inclusion.

- Time was shown to have two effects on the soils consisting of 60% moisture. An increase in curing time resulted in greater strength reductions being observed in areas surrounding the inclusion; as a result of further moisture build up. However at increasing radial distances, the curing time can be seen to enhance the strength properties of the soil, with more and more water being drawn towards the inclusion resulting in a dewatering effect. An increase in time can therefore be both beneficial and detrimental to the clay strength when utilising dry cement inclusions. Further study to examine ways in which this technology can be applied to soils at this moisture intended to benefit the whole system should be undertaken; however this is out with the scope of this present study.

- Incorporating a dry cement inclusion into laboratory prepared kaolin clay of 40% moisture has proven to increase the strength of the soil within the contents of this project of work. The largest reduction in moisture, hence increase in strength, is consistently observed in areas immediately surrounding the soil-inclusion interface, with a reduction in moisture change experienced at increasing distances from the inclusion.

- An increase in curing time is shown to have favourable effects on soils consisting of 40% moisture, with a continual reduction in moisture recorded in the clay sample. It was also shown that the curing time affects the inclusions radially influence; as a reduction in moisture is observed at greater radial distances from the inclusion with increased curing time.

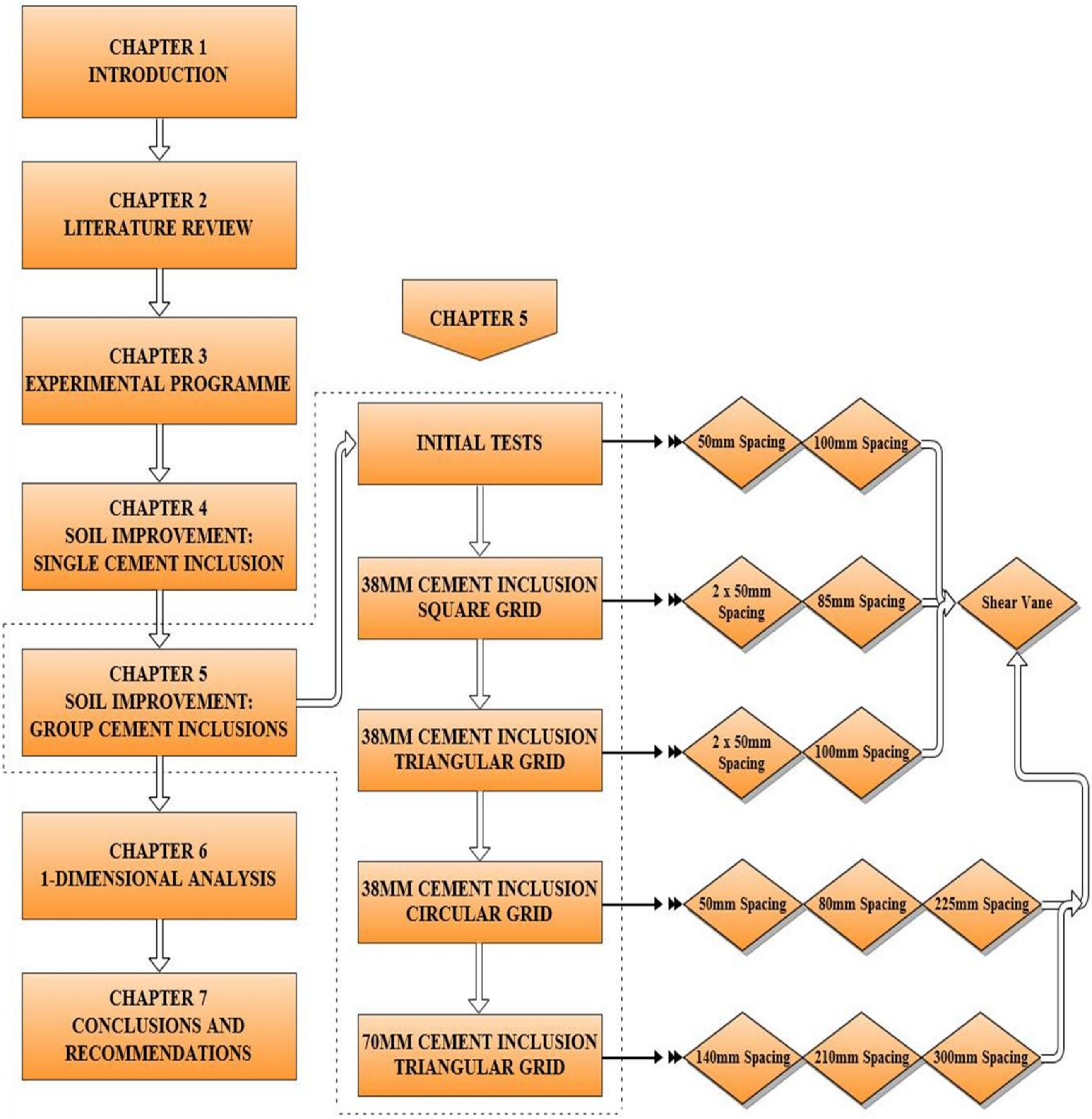
- The radial influence of the dry cement inclusions, in terms of strength improvement at increasing radial distance from the inclusions interface, is also influenced by the inclusion diameter. This is believed to be a direct result of the increased cement content, necessary to form the larger inclusions, requiring a greater volume of water absorption to satisfy the water demand of the cement. One disadvantage to increasing inclusion diameter is that an increase in shrinkage is likely to ensue.

- The formation of tensile cracks is likely to occur in soils of high moisture content as a result of the rapid hydration of the cement in these soil conditions. Tensile cracks are also a risk with increased cement diameter and cement content.

- Hydration of the cement to the core of the inclusion is achievable in both soil conditions and with increased inclusion diameter. This highlights that the system does not limit the ingress of moisture once the skin of the inclusion has sufficiently hardened (as was feared prior to testing). However, increasing the inclusion diameter does require a longer curing time for the core to hydrate to the same degree as the interface.

- Water has the ability to penetrate and react with the cement at the centre of the cement inclusion in all samples cured in clay of 40% moisture. It has been suggested that meniscus forces are responsible for continued hydration at the core of the inclusion, based on the skin and colour patterns observed during testing.

- Replacing Portland cement with CSA to form the inclusions has both physical and environmental advantages. The reduction in moisture can be shown to be greater for the CSA cement in comparison to the PC, 24 hours after installation (CSA provides a strength improvement 130% greater than that achieved by PC). This suggests that rapidly dehydrating the soil, in order to achieve consolidation, is more beneficial with larger strength improvements being achieved. However, further research should be undertaken with respect to this section of work.



> "I have had my results for a long time: but I do not yet know how I am to arrive at them"
>
> **Carl Friedrich Gauss (1777-1855)**

CHAPTER 5

SOIL IMPROVEMENT:

GROUPED CEMENT INCLUSIONS

5.1 INTRODUCTION

Having established the local dehydration effect of a single inclusion in Chapter 4, limited work was carried out to determine the significance of an area effect when employing dry cement inclusions in a number of group arrays. The research focused on analysing the performance of cement inclusions in clay soil of 40% moisture (50kN/m^2) with an inclusion diameter of 38mm used for the initial tests; as this limited the volume of hand mixed kaolin required for testing. The inclusions were installed for a period of 28 days and left to interact with the pore water; based on the results from Chapter 4, with moisture content samples and hand shear vane tests being performed. From these results recorded, the most effective group array was carried forward for a further investigation involving inclusions of 70mm diameter.

This Chapter discusses the most effective array of grouped inclusions in order to achieve the maximum strength improvement possible, with references to established techniques and their adopted group arrays.

5.2 INITIAL TESTS

Before testing different group arrays, it was deemed necessary to isolate two inclusions in order to study their mechanical interaction with one another, when placed at different lengths apart. The purpose for this was to allow any limitation to the spacing between inclusions to be quickly identified. It was the intention of the Author to place the inclusions at some distance apart to generate an 'overlapping' dehydration effect to be experienced in the soil, i.e. an area of soil influenced by both inclusions desire to draw in water as shown in Figure 5.1.

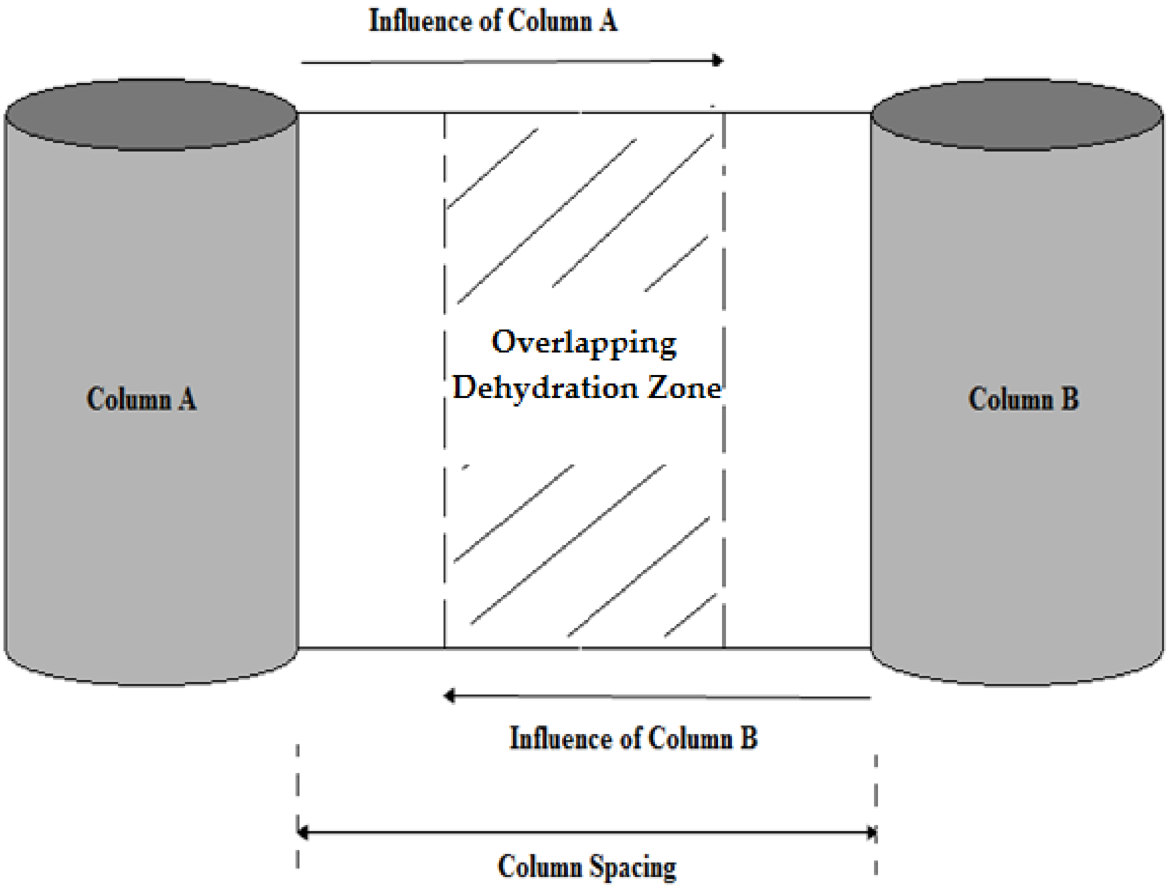


Figure 5.1: Overlapping dehydration influence of two inclusions

Arbitrary spacing values of 50mm and 100mm were used during the initial tests. These values were selected based on the local dehydration effect of a single 38mm inclusion being shown to reach the full 75mm distance from the inclusion after 28 days in Chapter 4, thereby an 'overlapping' effect was most likely to occur. The following moisture results from the initial tests can be viewed in Figures 5.2 and 5.3.

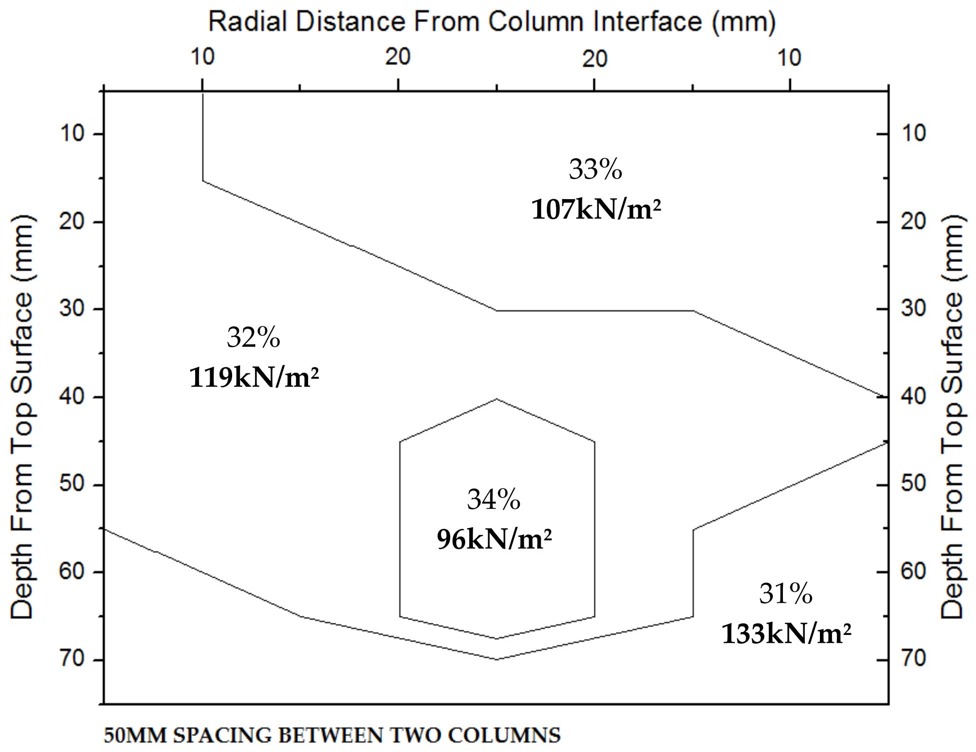


Figure 5.2: Moisture content between two 38mm cement inclusions of adopted 50mm spacing

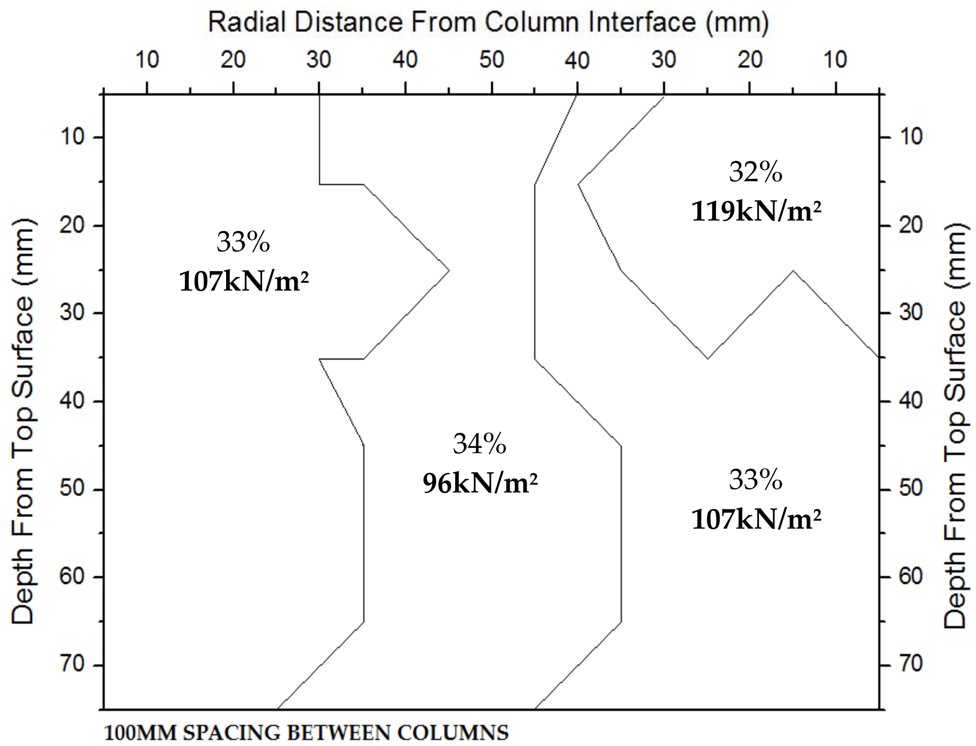


Figure 5.3: Moisture content between two 38mm cement inclusions of adopted 100mm spacing

From these results, it is apparent that the soil experiences a strength improvement in both cases; however the magnitude of improvement can be shown to be greater in the 50mm spaced sample in comparison to the 100mm. As the weight of water absorbed by each of the four inclusions was approximately the same, 50-53% by weight of dry cement, this has been accredited to the two inclusions in the 50mm sample dewatering the soil in a closer vicinity to one another. This was expected prior to testing, as the combination of the pore water experiencing a greater pulling force towards either of the two inclusions, coupled with a reduced travel distance to the inclusion, would ultimately lead to a larger dehydration effect to be experienced by the soil. Similarly as in Chapter 4, TGA results from the four inclusions show that the central core of the inclusion has hydrated to the same degree as cement at the interface.

The main points taken from these tests where: a greater dehydration effect could be experienced with a reduced distance between inclusions and the interaction between two inclusions is not limited to a radial distance of 100mm when using 38mm inclusions. For these reasons, the Author was not concerned with adopting spaces between 50mm and 100mm for any the three group arrays investigated in the following section.

5.3 ARRAY 1 (SQUARE GRID)

The first array of grouped cement inclusions selected for testing reflected the square grid array commonly adopted for Controlled Modulus Columns in practice. Six 38mm inclusions (area ratio 0.23) were cast in a 390 x 300mm container with a spacing of 50mm adopted between inclusions (1.3 times the diameter of inclusion) set up as shown in Figure 5.4.

In order to understand the interaction between the inclusions within this array, three separate locations were selected for extracting moisture samples (indicated in Figure 5.5), with shear vane tests performed in undisturbed areas as a rough confirmation of strength improvement.

Figure 5.4: Square grid array of 38mm cement inclusions

(Ruler and key shown for scaling purposes)

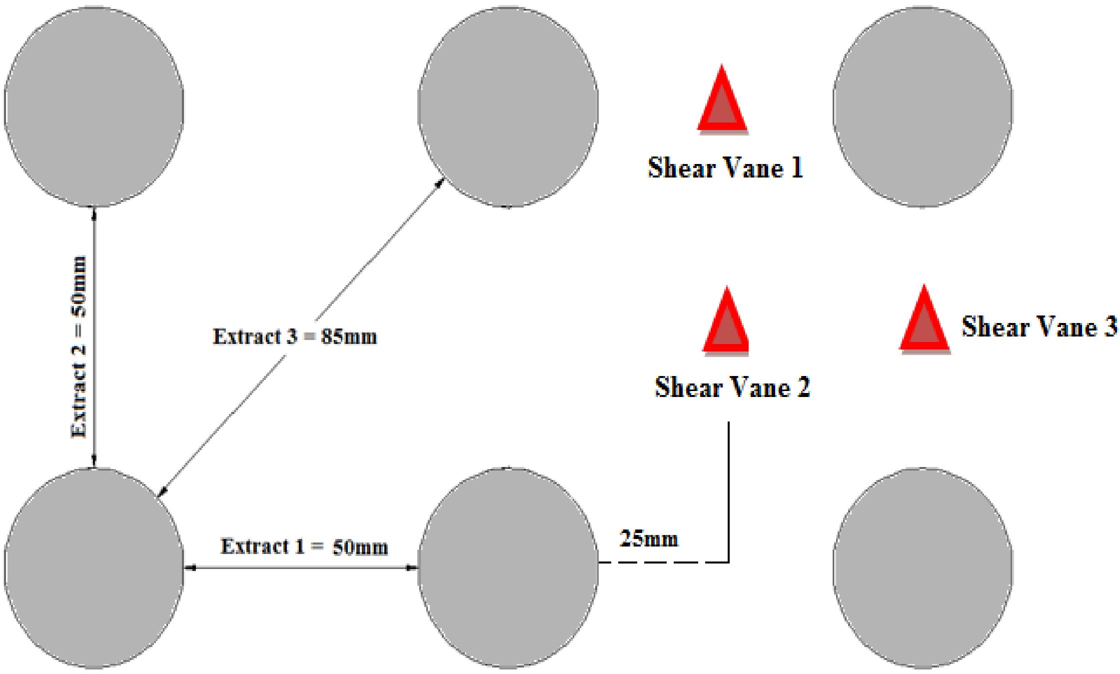


Figure 5.5: Position of moisture samples and hand shear vanes in square grid array

The results for the moisture tests can be viewed in the Figure 5.6:

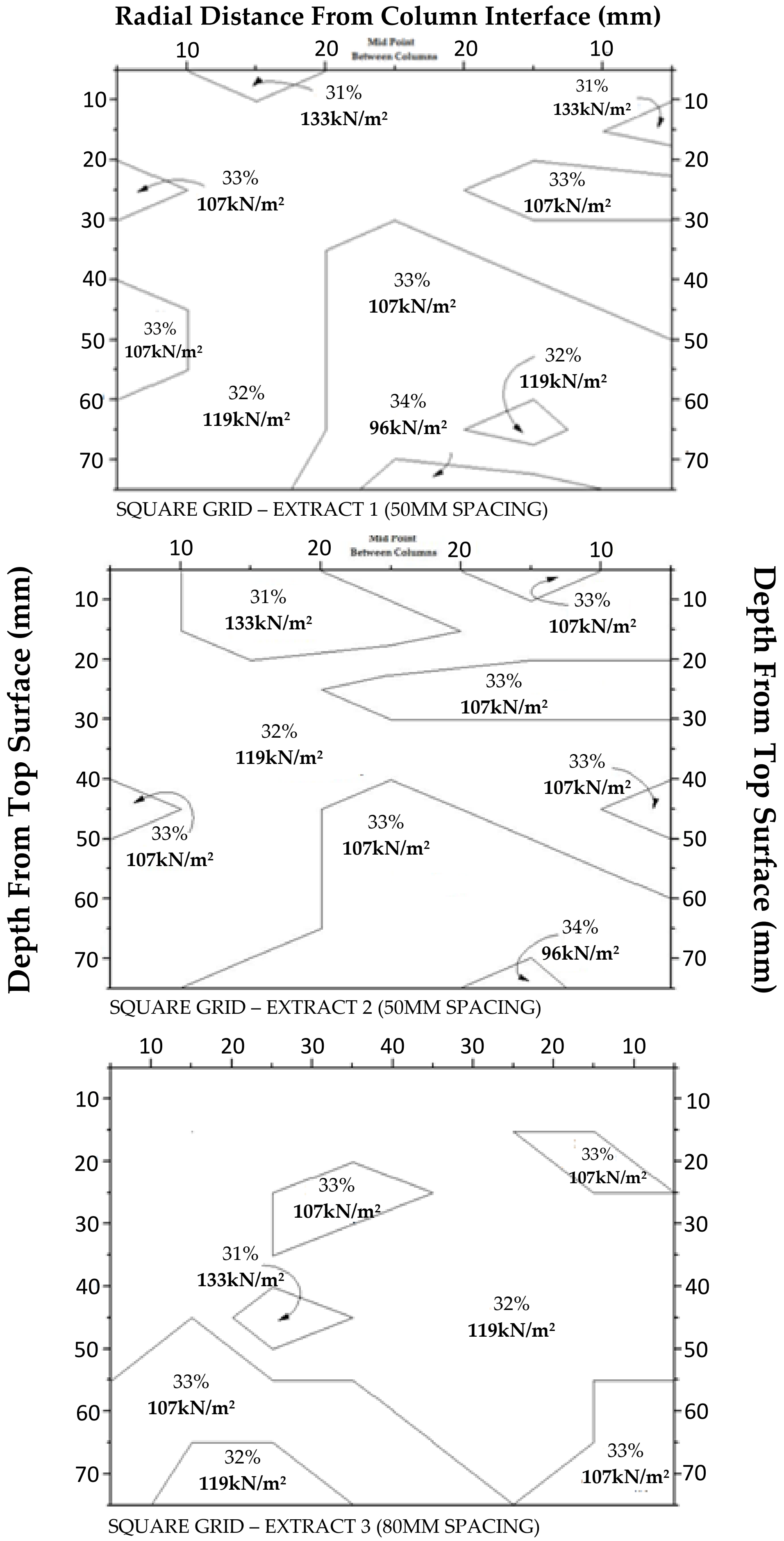


Figure 5.6: Moisture content results taken from 3 locations in square array

From these results it is clear that the interaction of inclusions grouped together in this array have a far greater influence on the dehydration of the soil in comparison to a singular inclusion at 28 days, with larger moisture reductions recorded. Surprisingly, the largest moisture drop was not recorded at the interface as was the case with all singular inclusions. This is due to the local effect of both inclusions 'overlapping' to provide an area effect, which was intended prior to testing. The largest improvement was recorded as 31% (133kN/m^2) which corresponds to a strength increase of 167% in comparison to the clay if left untreated. The results from the shear vane confirm that strength has increased significantly as a result of the inclusion array utilising pore water from the soil.

Table 5.1: Hand shear vane results for square grid array

Vane Position (Figure 5.5)	Shear Strength (kN/m^2)
Control Sample	48
Shear Vane 1	118
Shear Vane 2	125
Shear Vane 3	120

After performing the appropriate tests on the clay soil, the hardened inclusions were removed from the clay and weighed. Each inclusion was shown to have absorbed 52 -54% water, by weight of dry cement, which was similar to the weights noted during the initial tests in section 0. TGA was carried out on three of the hardened samples in the same manner as in Chapter 4, with all three inclusions indicating that water had constant access to the inclusion core; as the degree of hydration was the same throughout the entire cross sectional areas for each of the inclusions.

5.4 ARRAY 2 (TRIANGULAR GRID)

The triangular grid array was selected as it is commonly adopted for deep soil mixing operations as discussed in the literature. The test was also performed in a 390x300mm container and consisted of eight 38mm inclusions (area ratio 0.31), with both 50mm and 100mm (1.3 to 2.63 times the inclusion diameter) spacings adopted between inclusions (Figure 5.8).

Figure 5.7: Triangular grid array of 38mm cement inclusions

(Ruler and key shown for scaling purposes)

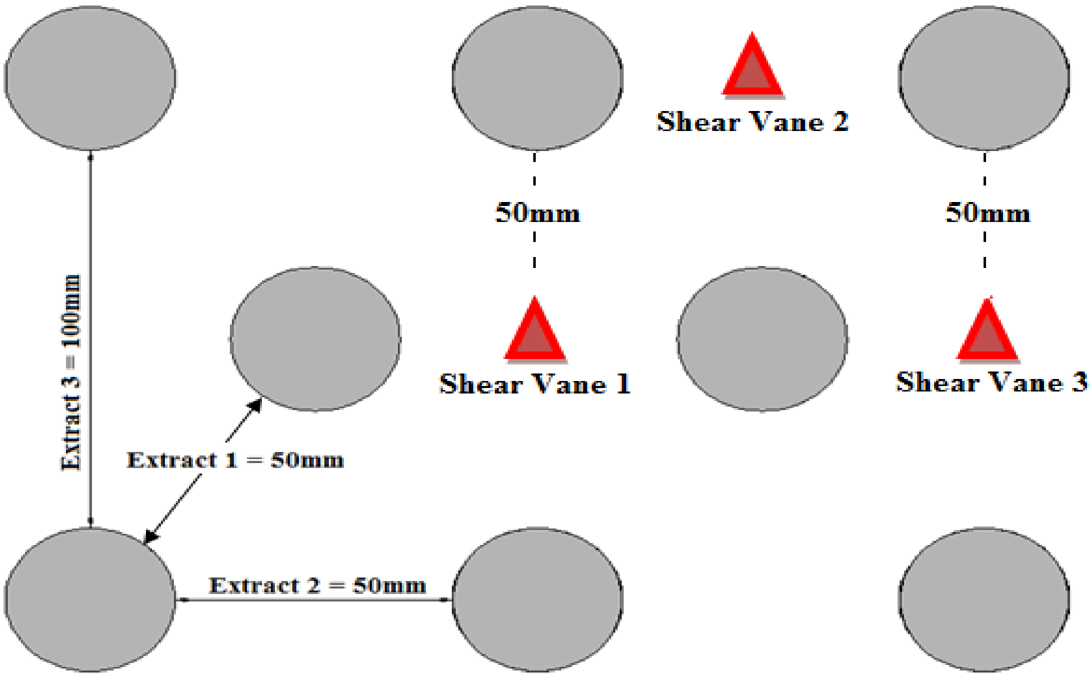


Figure 5.8: Position of moisture samples and hand shear vanes in triangular grid array

The exact same procedure was performed in this array as in the square grid, with the results provided in Figure 5.9.

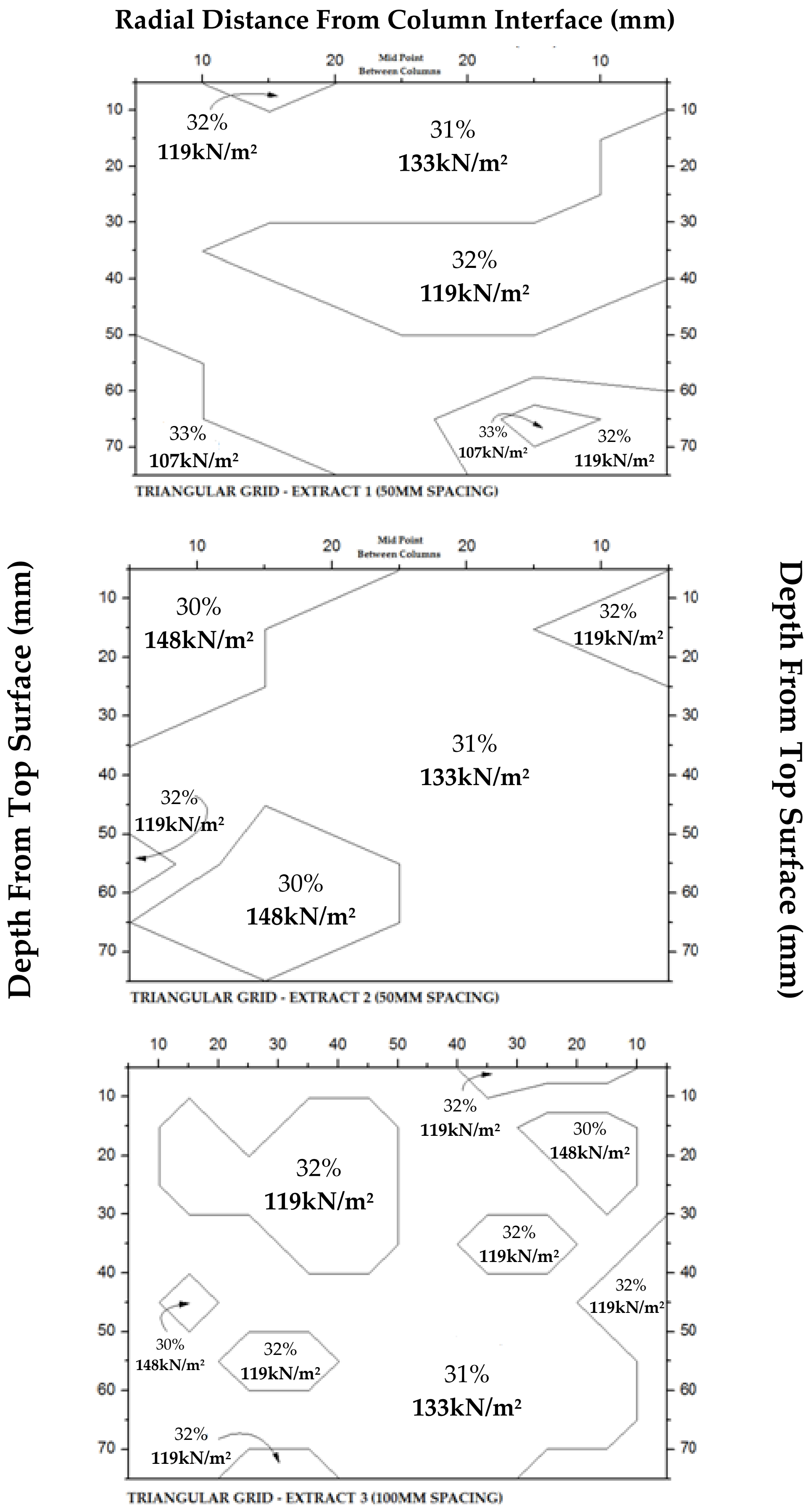


Figure 5.9: Moisture content results taken from 3 locations in triangular array

From these results, it is shown that the overall improvement in shear strength is greater for this array than in the square grid, with the majority of the three sample positions consisting of 31% moisture (133kN/m^2 strength). It can also be observed that the moisture content has been lowered to 30% (148kN/m^2) in some locations, which corresponds to a shear strength improvement of 198% in comparison to the untreated clay. This is likely to be the result of an increase in cement content; leading to further increase in water absorption, as eight inclusions were used in this case, instead of the six adopted in the square grid. Again the hand shear vane tests provide a rough indication of strength increase, as shown in Table 5.2.

Table 5.2: Hand shear vane results for triangular grid array

Vane Position (Figure 5.7)	Shear Strength (kN/m^2)
Control Sample	**48**
Shear Vane 1	**127**
Shear Vane 2	**131**
Shear Vane 3	**142**

Each hardened inclusion showed that between 50-54% of water absorption, by weight of dry cement had occurred, with TGA confirming that three of the inclusions tested did allow water to continually access the core of the inclusion.

5.5 ARRAY 3 (CIRCULAR GRID)

The final array under investigation was a circular grid which consisted on eight 38mm inclusions (area ratio 0.31). This array was an adaptation to the triangular arrangement investigated in Figure 5.4, and was intended to provide larger spacings between certain inclusions. This should determine if improvement could be experienced over a larger spatial distance than 100mm. The layout of the array and the positions selected for moisture and shear vane testing can be viewed in Figures 5.10 and 5.11 respectively.

Figure 5.10: Circular grid array of 38mm cement inclusions

(Ruler and key shown for scaling purposes)

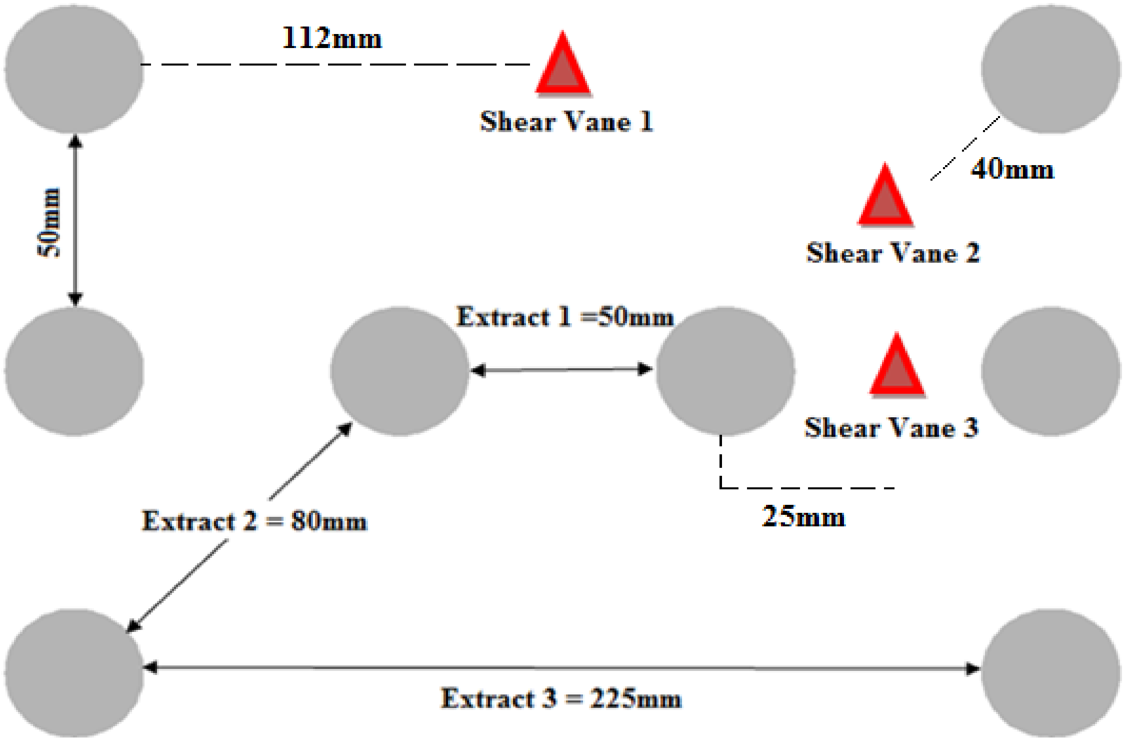


Figure 5.11: Position of moisture samples and hand shear vanes in circular grid array

The results recorded for this test can be viewed in Figure 5.12 as follows.

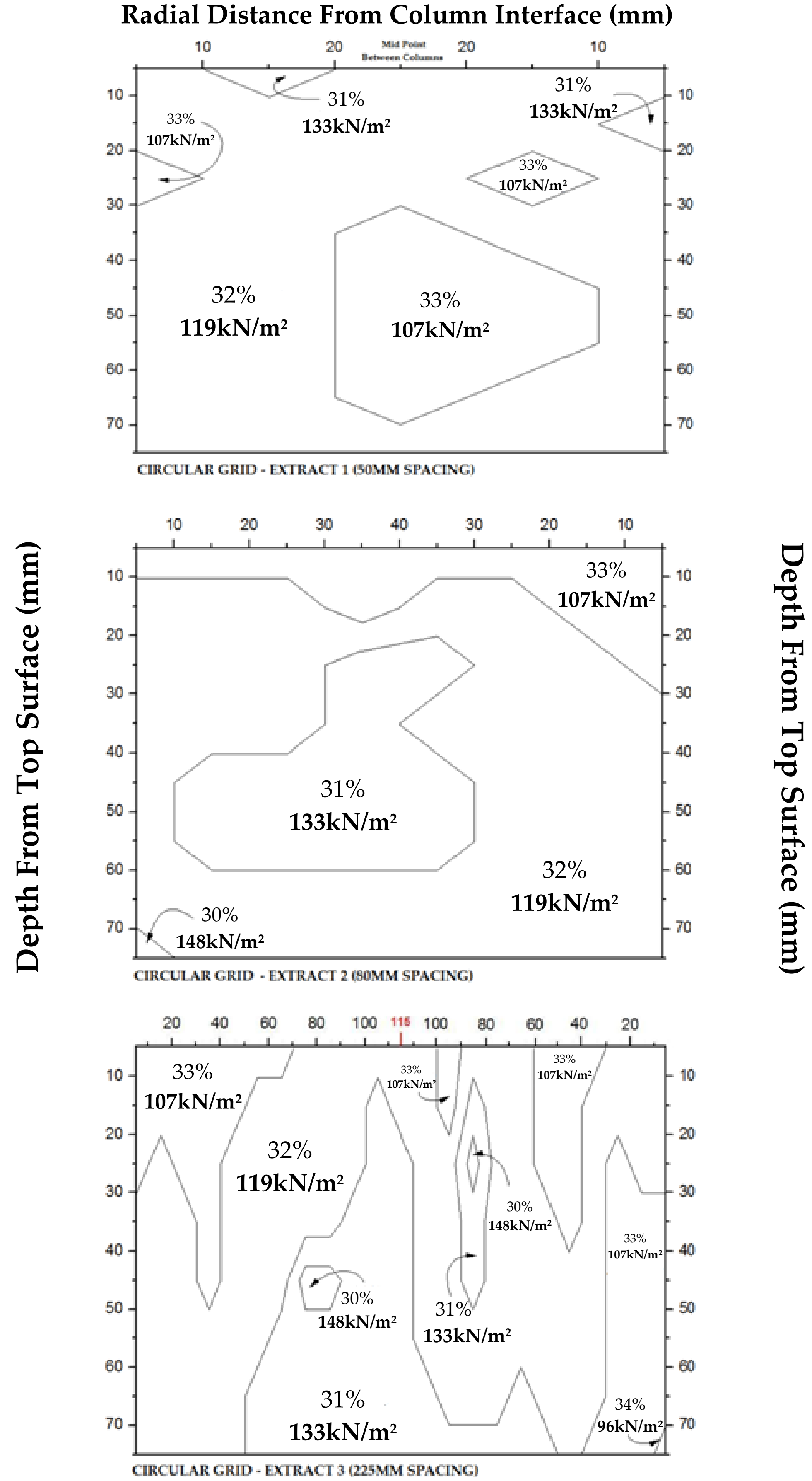


Figure 5.12: Moisture content results taken from 3 locations in circular array

From the results of the circular grid arrangement it is evident that a significant overall improvement to the clay soil occurred between inclusions, with the majority of the strength improvement as much as 139% (116kN/m^2) in comparison to the control sample. However, the triangular grid outperforms this with a majority improvement of 167% (133kN/m^2). Never the less this array provides evidence that the spacing between inclusions can reach distances of 225mm whilst maintaining a high degree of improvement to the surrounding. Shear vane test results can be viewed in Table 5.3.

Table 5.3: Hand shear vane results for circular grid array

Vane Position (Figure 5.11)	Shear Strength (kN/m^2)
Control Sample	**48**
Shear Vane 1	**125**
Shear Vane 2	**128**
Shear Vane 3	**115**

The hardened inclusions again showed that water absorption was in the region of 51-54%, by weight of dry cement, as was the case for the other arrays. However, surprisingly TGA results indicate that only two of the three inclusions had access to water at the core of the cement inclusion; by way of the degree of hydration at core equalling the cement at the interface. The Author is at a loss to explain why this was the case, as the inclusion appeared to display the same physical characteristics throughout its cross section as all other hardened inclusions at 28 days. Possible reasons could have been a faulty test or contamination of the cement during preparation for TGA testing.

5.6 70MM DRY INCLUSION DIAMETER (TRAINGULAR GRID ARRANGEMENT)

From the results in section 5.5, the triangular grid was considered to be the most effective array to carry forward for an analysis involving inclusions of 70mm diameter. This decision was based on the array providing consistent moisture results whilst also producing the most significant moisture reduction of 10% in some locations of the clay.

The test was performed in a container of 500 x 380mm in size with five 70mm inclusions used (area ratio 0.64).

From the results recorded in Chapter 4 for single inclusions of 70mm diameter, the decision was taken to adopt spacings of 210mm (3 times the inclusion diameter), 140mm (2 times the diameter) and 300mm (4.3 times the diameter) between inclusions (as shown in Figure 5.13).

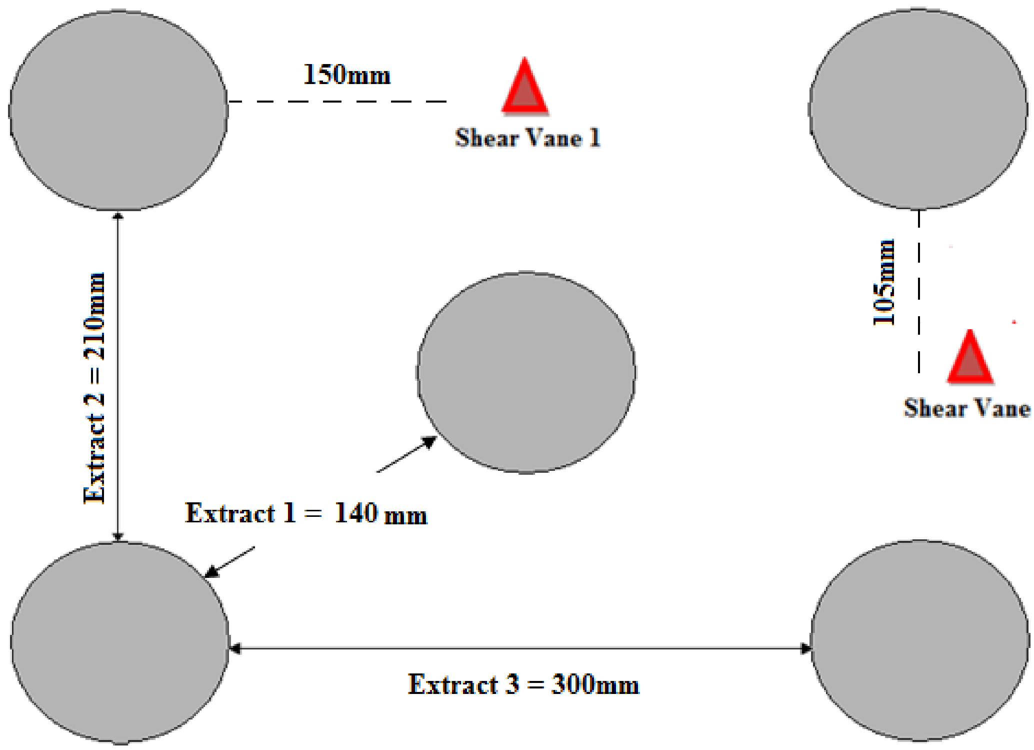


Figure 5.13: Triangular array adopted for 70mm inclusions

Extracting moisture samples from these positions allowed the results presented in Figure 5.14 to be produced.

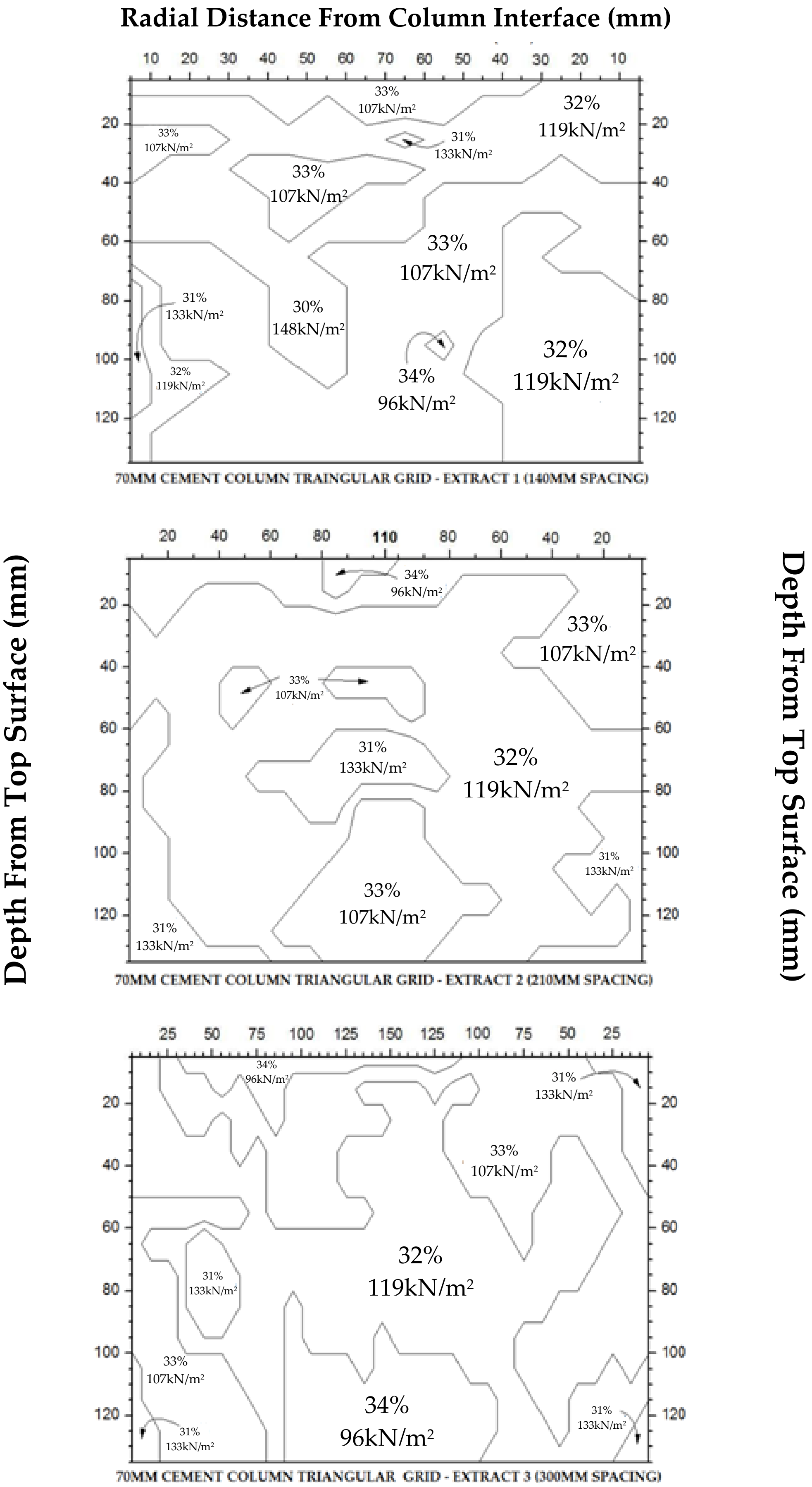


Figure 5.14: Moisture content results from 70mm triangular array

The results recorded in Figure 5.14 indicate that an increase in inclusion diameter can allow larger spacings to be adopted between inclusions, with a high level of moisture change still provided. Again, hand shear vane tests were performed to enhance the findings from these moisture content results. These were performed in the positions indicated in Figure 5.14 with Table 5.4 providing the results obtained.

Table 5.4: Shear vane results for 70mm triangular grid array

Vane Position (Figure 5.5)	Shear Strength (kN/m^2)
Control Sample	**48**
Shear Vane 1	**110**
Shear Vane 2	**118**

The hardened inclusions were weight at the end of the 28 day test with water absorption shown to be between 55-61%, by weight of dry cement. TGA analysis performed on three of the inclusions indicated that hydration was the same at the inclusion core as the cement at the interface, thereby enhancing the findings in Chapter 4.

5.7 CHAPTER SUMMARY

From the results recorded in this section of work the following conclusion can be drawn by the Author:

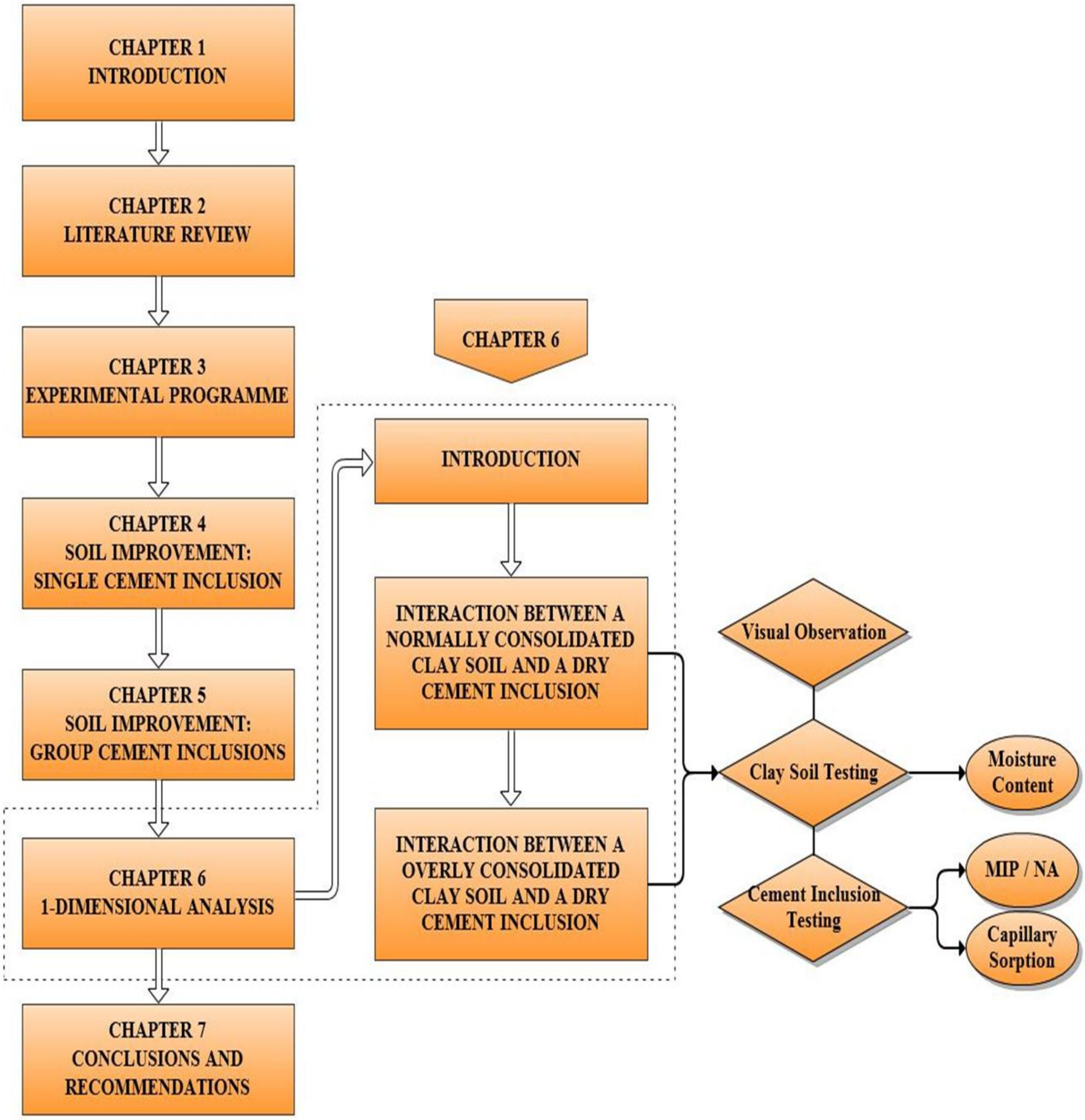
i. The interaction between two dry cement inclusions can have additional dehydration area effects in the soil. The position of the inclusions relative to one another influences the magnitude of improvement; with smaller spacings between inclusions providing greater increases in strength. This is the result of 'overlapping' local dehydration effects as depicted in Figure 5.1.

ii. Increasing the area replacement ratio can have a beneficial influence on the clay soil; with increased magnitudes in strength improvement recorded. However the array arrangement is also an influential factor.

iii. Adopting larger inclusion diameter can increase the space necessary between inclusions. This is the result of an increase in cement content absorbing a greater volume of water in order to satisfy the cement affinity for water.

iv. All cement inclusions, with the exception of one, enhance the TGA finding in Chapter 4; with the central core of the inclusion shown to have the same level of access to the soils pore water as cement at the interface.

v. Inclusions can absorb in excess of 50% water, by weight of dry cement, when left to interact with the pore water up 28 days. This enhances the findings from Chapter 4.



"We must reinforce arguments with results"

Booker T. Washington (1856-1915)

CHAPTER 6
1-DIMENSIONAL INVESTIGATION OF EFFECT ON SOIL PORE WATER CONTENT

6.1 INTRODUCTION

In this Chapter, the results from a series of 1-dimensional investigations are presented. The focus of these investigations were on the ability of a dry cement layer to utilise pore water from kaolin clay soil; subjected to a preconsolidation stress of 140kPa. With the use of a specially designed compression rig (detailed in 3.6.3), visual inspections and physical tests were performed on normally consolidated clay samples (OCR 1.0) at 1, 3, 7, 14 and 28 days. Tests carried out in the clay were in the form of moisture content samples; extracted at increasing distances from the cement-soil interface and hand shear vane tests. Tests performed on the hardened cement included Capillary Sorption, Thermogravimetric Analysis (TGA), Mercury Intrusion Porosimetry (MIP) and Nitrogen Adsorption, all with the aim of assessing the cement layers performance over time.

Other tests, focused on the ability of cement to utilise pore water from overconsolidated kaolin clay samples, with the same tests described above being performed. Arbitrary OCR values of 1.5 and 2.0 were selected prior to testing, with tests performed at an arbitrary time of 7 days after the cement was first placed in contact with the clay.

It was important to know how the system proposed in this work, was likely to perform in the event of a possible worst case scenario. With this in mind, a final set of tests were conducted which aimed to provide some insight into the cements behaviour with a clay soil experiencing a reduction in confining stress. This was put forward as the worst case scenario, as during installation it is possible that stresses in the surrounding soil will relax as a result of the cement not providing the same level of confinement as the soil removed to facilitate the inclusion; in the same way an excavation results in the relaxation of stresses in the surrounding soil. This could potentially limit the interaction between the pore water in the soil and the cement; resulting in reduced moisture changes and hence reduced improvements in soil strength being experienced in the surrounding soil.

6.2 INTERACTION BETWEEN A NORMALLY CONSOLIDATED CLAY AND A DRY CEMENT INCLUSION

Prior to testing, three samples of kaolin were mixed to 120% moisture content and consolidated to 140kPa, as detailed in section 3.6.2. These were used as controls to ensure that the consolidated samples displayed consistent moisture results along their length, whilst also allowing a visual inspection to verify that there were no air voids trapped within the sample by consolidating in this manner. Each sample removed from the consolidation cell was approximately 170mm in height. However as the ends of the sample were extremely wet in comparison to the rest of the sample, 10mm was removed from either end. This provided samples of 150mm height, with moisture contents averaging 54 ± 3%.

As described in 3.6.2, the samples were consolidated to 140kPa for a period of 5 days before being transferred to the compression rig. At which point a 60mm layer of dry cement, of known weight, was brought into contact with the underside of the kaolin sample (i.e. the base of perspex viewing tube). A non-permeable piston was then used to load the clay back to a normally consolidated state. The load was applied to the sample for the duration of the test time (i.e. 1, 3, 7, 14 and 28 days) with the perspex tube allowing a visual inspection to be performed.

6.2.1 Visual Inspection Performed During Testing

A visual inspection involving the penetration of water through the cement layer (indicated by progressive darkening of the cement) was performed over a range of arbitrary time intervals up to a time of 24 hours (Figure 6.1).

From this, it can be shown that the pore water in the soil is rapidly absorbed by the dry cement layer when initially brought together in contact; as a result of cements hygroscopic nature. It is suggested that this initial water absorption causes a concentration difference between the newly wetted cement at the surface and the dry unhydrated cement near the base of the perspex tube to form; causing the main transport mechanism of water through the cement layer to change from absorption to diffusion.

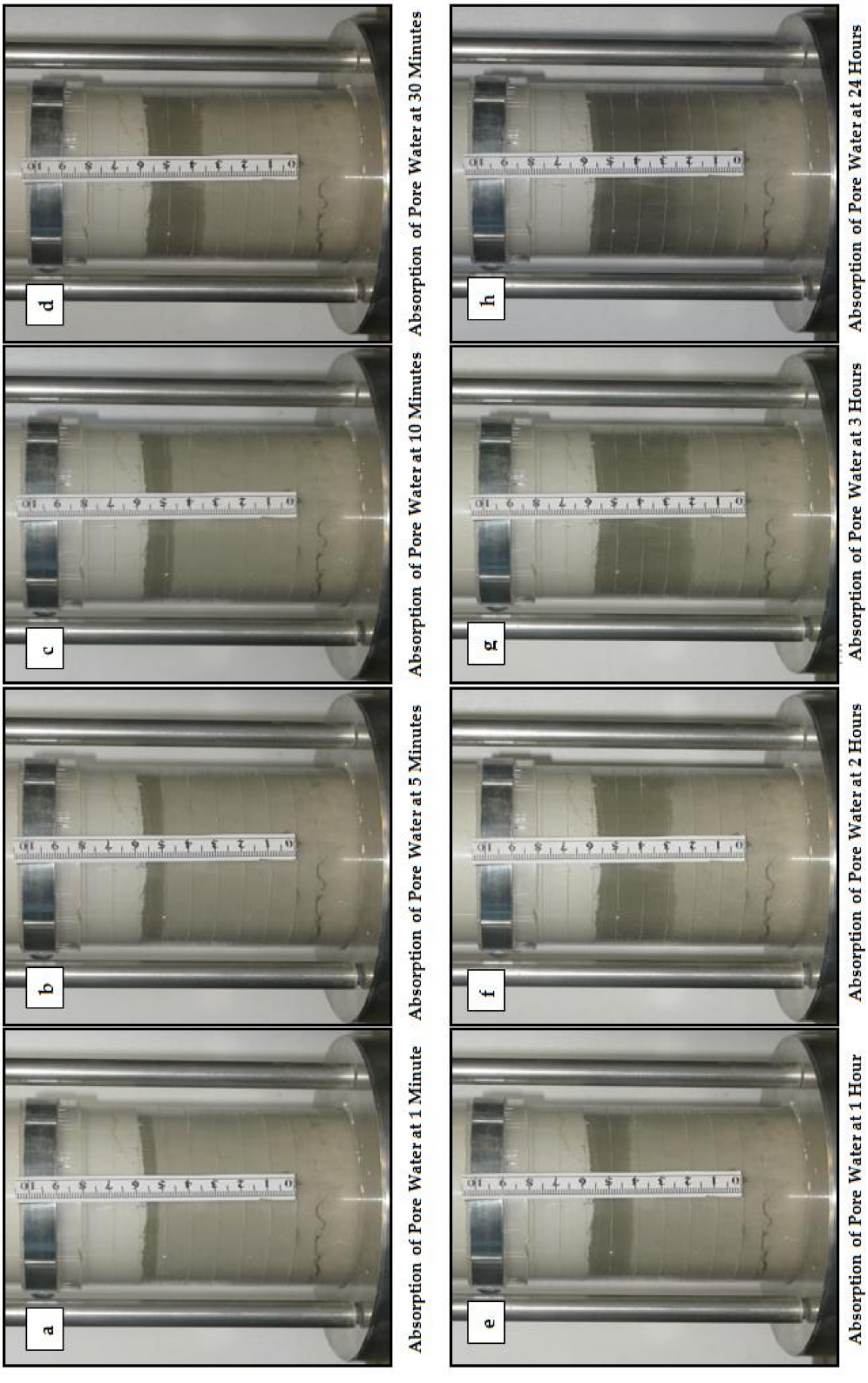


Figure 6.1: Visual representation of pore water absorption into cement layer (Darkened cement indicates presence of water)

The water uptake, i.e. the depth of water penetration through several cement samples, was plotted against the square root of time (Figure 6.2) up 3 hours after initial contact between the cement and pore water.

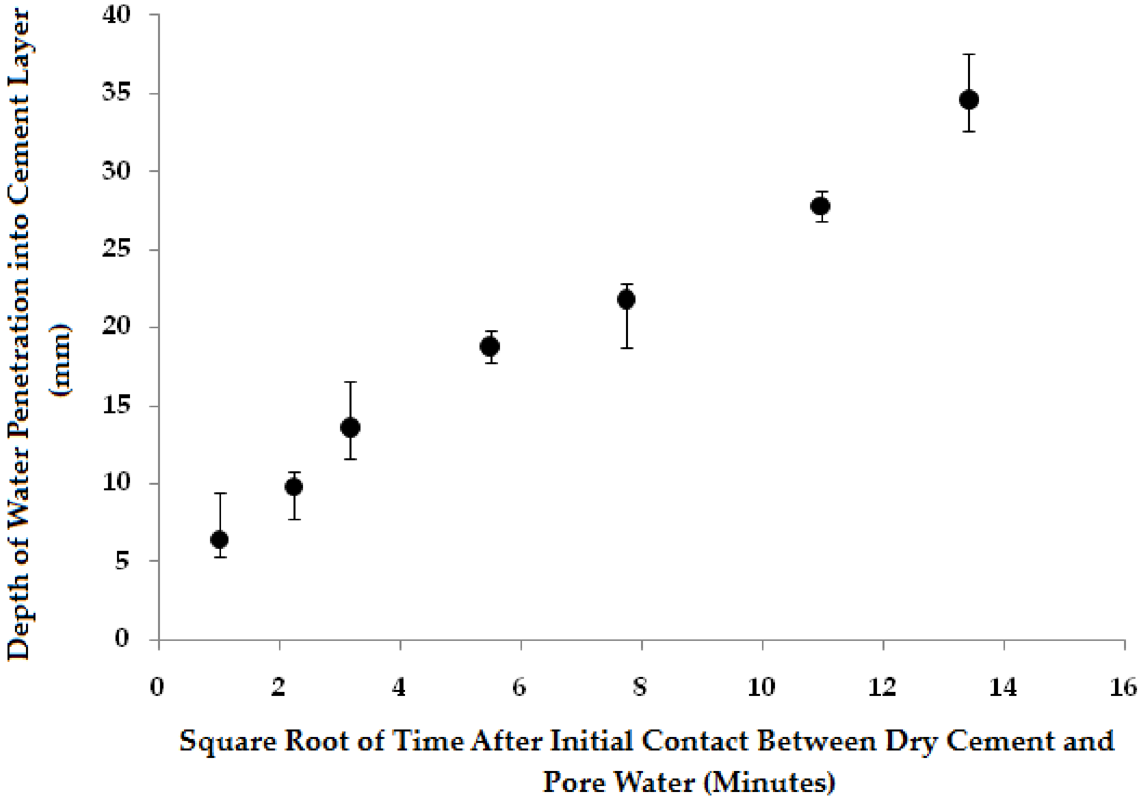


Figure 6.2: Water penetration through cement layer over time when in contact with a normally consolidated soil

After a period of approximately 3 hours, the absorption of water into the cement layer becomes increasingly difficult to determine through visual inspection, as the absorption rate drops off sharply. This has been attributed to the cement at the interface beginning to harden around this time based on the paste setting tests; where results show that at 3 hours the 300 gram weight needle was unable to penetrate samples to depth greater than 1mm, as detailed in section 3.7.2. It is suggested that this hardening process reduces the volume of water absorbed by the cement; however leads to the formation of capillaries which over time develop and generate tensile forces. These tensile forces allow the cement to continually attract and absorb water, albeit at an increasingly reduced rate over time. This was also found to be the case in Chapter 4.

From Figure 6.2, the initial rate of water absorption was determined from the slope of the line and was found to be approximately 2.1 mm/minute$^{0.5}$.

6.2.2 Kaolin Clay Soil Testing

As mentioned, the ability of a dry cement layer to dewater the surrounding clay soil was determined through moisture content samples being extracted from the kaolin clay at increasing distances from the cement layer interface. Similarly to the tests performed in previous Chapters, the moisture content was determined using the oven drying method, with samples extracted in 10 x 10 x 10mm sizes after a period of 1, 3, 7, 14 and 28 days. Again the purpose of this was to examine if an increase in time has a beneficial influence on the moisture reduction, and hence strength increase, in the clay. The results of the moisture test are presented in Figure 6.3.

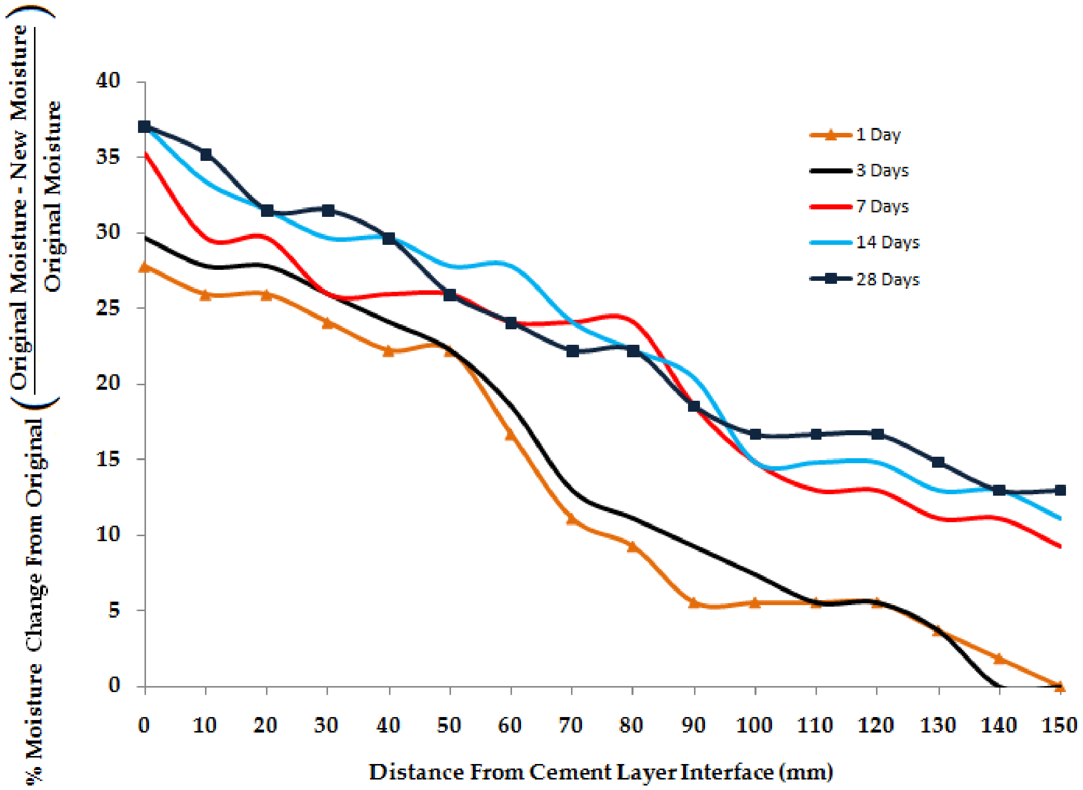


Figure 6.3: Moisture movement recorded in a normally consolidated soil over time

From the results recorded in Figure 6.3, it is clear that the introduction of a dry cement layer has a profound influence on the moisture of a normally consolidated clay layer, with moisture reductions recorded throughout the sample length. Again the maximum reduction in moisture can be shown to be at the cement interface for all test times, which is most likely the cause of rapid absorption of the pore water taking place in a zone immediately surrounding the cement. It is suggested that as water is absorbed, a pressure gradient is induced in the soil drawing water from increasing distances to migrate towards the cement, which as a result causes a reduction in moisture to be experienced at increasing distances from the interface.

However, Figure 6.3 seems to suggest that there is little further moisture reduction experienced in the soil after a period of 7 to 14 days i.e. the system seems to shut down. This is not consistent with the behaviour of the clay tested in Chapter 4; as increased contact time was shown to benefit the system with further moisture reductions recorded at increasing radial distances from the cement interface. It is possible that the difference in results is due to the perspex tube limiting the cement surface area exposed to the soil, i.e. the cement in Chapter 4 would be closer to the soil at any one point in the inclusion and would have more access to the pore water; resulting in further moisture changes over time.

From Figure 6.3, it can be shown that the total depth of moisture change exceeds the 150mm sample length for clay left to interact with the cement for more than 3 days, i.e. the soil does not equilibrate. By providing a best fit line between the 14 and 28 day moisture results (Figure 6.4), it can be estimated that a total distance of 210mm from the cement interface is likely to experience a change in moisture (and as a result benefit from some level strength improvement).

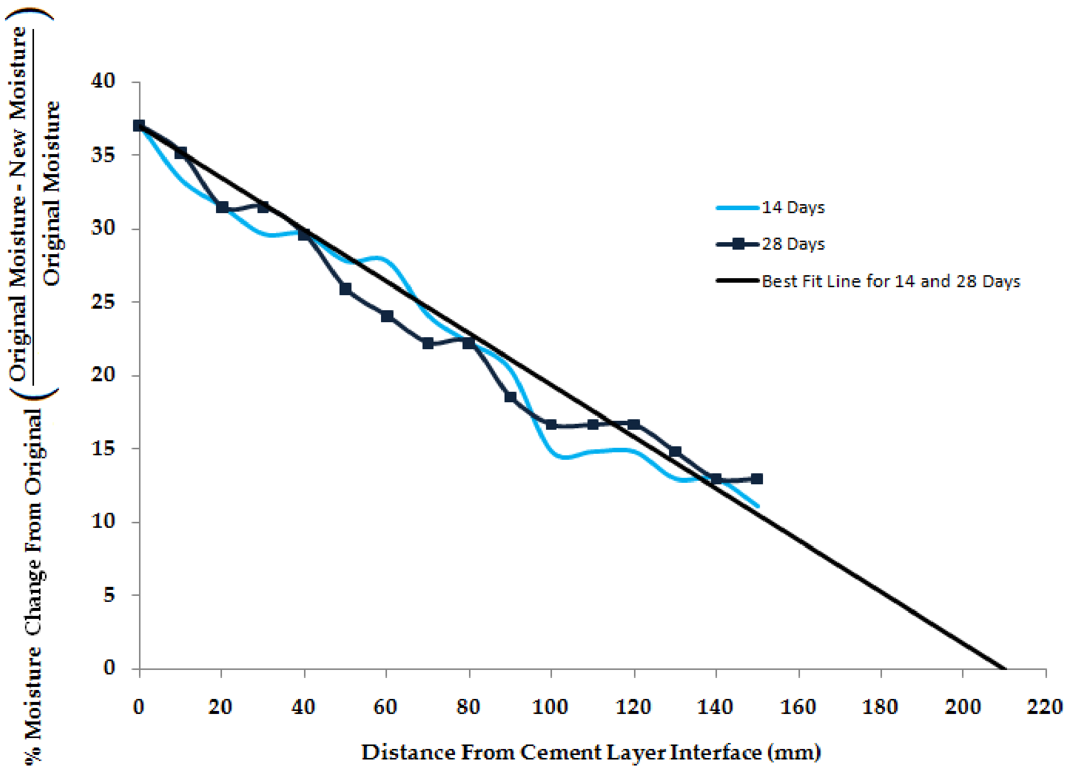


Figure 6.4: Estimated total depth of moisture content change from cement layer

As well as extracting moisture samples, hand shear vane tests were performed in the kaolin samples in order to provide a rough indication of whether or not an improvement in soil strength had resulted from the soil dehydration. The intention was for the results to provide a profile of shear strength at various distances from the cement interface. Unfortunately, the results from these tests did not allow such a profile to be generated as the shear strength value remained constant at all depths for each of the clay samples tested. This is most likely the result of the shear vane not being sensitive enough to detect the changes in strength observed in the soft soil. However, from the results it can be shown that the shear strength improves with an increase in time (Table 6.1).

Table 6.1: Hand shear vane results in 1-D compression rig

Length of Test Time, Days	Hand Shear Vane Results, kPa
Control Sample	17
1	22
3	27
7	31
14	42
28	46

6.2.3 Hardened Cement Inclusion Analysis

Similarly to Chapter 4, the weight of dry unhydrated cement necessary to form the cement layer was weighed prior to testing, with the weight of water absorbed determined by the weight increase in the sample after each test. From this, it was determined that water absorption continues to occur for the full duration of the test period; with the cement shown to have absorbed 42% water, by weight of dry cement, after a 24 hour period. This value increased to 45% by 3 days, with 48% and 49% recorded at 7 and 14 days respectively. The weight of water absorbed at the end of the 28 day curing period was recorded as 53%.

TGA testing was performed on the hardened cement at each stage of curing, with samples extracted every 10mm from the cement-soil interface. Figure 6.5 contains the results.

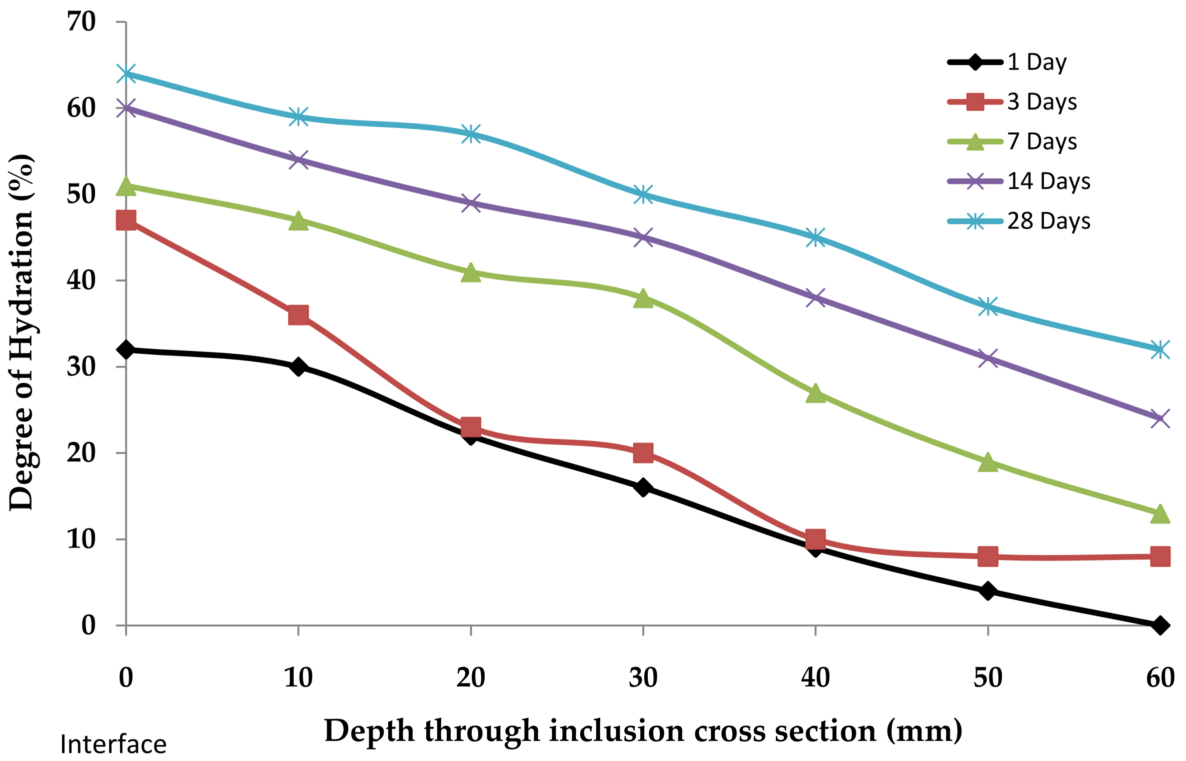


Figure 6.5: TGA results at increased depths from the cement interface at each test time

From these results, it is apparent that the cement at the interface has a greater interaction with pore water than the cement at increasing distances from the interface; with the degree of hydration higher for all test times. Further evidence of this was provided in the colour profile generated throughout the hardened cement, with darker shades of grey noticeable at the interface indicating that the cement had more access to water (as shown in Figure 6.6).

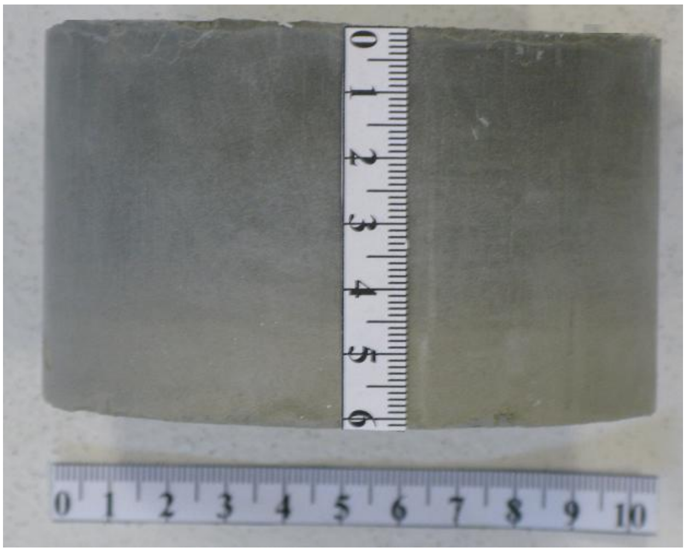


Figure 6.6: Hardened cement in contact with a normally consolidated clay for 28 day

It is also apparent that the cement at increasing depths from the interface has continual access to water; with the degree of hydration shown to increase with an increase in time. This would suggest that pore water is continually being absorbed by the cement; however, this is not consistent with the moisture profile generated in Figure 6.3 with the system appearing to shut itself off from any moisture changes after a time of 7 to 14 days. It is believed that meniscus forces discussed in Chapter 4, i.e. the 'onion skin' effect, are the reason behind the difference in the behaviour of the cement in relation to the moisture changes after 7 to 14 days. This is thought to be the case as cement hydration is shown to continue at increasing depths into the layer, even after the system shuts off the removal of pore water from the soil; which suggests that water absorbed at the interface gradually migrates through the cement layer with time.

Unlike the hardened inclusion in Chapter 4, there was no evidence of these meniscus forces occurring in the cement layer (as the cement was extremely smooth with no visible sign of an 'onion skin' effect). This could be a result of the way in which the samples were loaded, as in the 1-D case there are high pressures involved in comparison to the containers used to analyse the single inclusions. However, another possibility could be that the cements interaction with water is restricted to a single surface, whereas in Chapter 4 the cement would have access to water from all locations surrounding the inclusion perimeter. This could also explain why the degree of hydration is not the same at any two depths in the cement layer.

Mercury Intrusion and Nitrogen Absorption Test Results

Mercury and nitrogen tests were performed on the hardened cement samples placed in contact with normally consolidated clay to 3, 7, 14 and 28 days. The intention was for these tests to enhance the findings from the TGA results and show that continued hydration takes place with time; as indicated by a reduction in the porosity of the cement. Control samples of Portland cement; mixed to a water/cement ratio of 0.5 and stored in a water tank set to 20^{o}C ± 2^{o}C for the required length of time, should allow the general behaviour of cement cured in a non-limiting environment, to be compared with the behaviour of the cement in contact with clay.

The decision was taken not to perform either of the porosity tests on the 1 day cement samples, as the risk was that the microstructure would not have developed sufficiently in time for testing. It was likely that the pressures involved in the test would break down the pore walls and overestimate the porosity, which would not be of any benefit to the test findings. The results are presented in Table 6.2.

Table 6.2: Continual development of cement pore structure as depicted from MIP and Nitrogen Adsorption testing

Length of Interaction with Water, Days	Total Porosity, %			
	Mercury Intrusion (MIP)		Nitrogen Adsorption	
	Control (w/c 0.5)	Cement Sample	Control (w/c 0.5)	Cement Sample
3	39.9	42.5	32.0	34.5
7	33.4	35.6	28.4	29.1
14	30.8	31.1	25.7	26.9
28	27.9	28.7	21.7	22.1

The results obtained in Table 6.2 indicate that as expected the porosity of the cement reduces with an increase in time; as a result of the hydrated product C-S-H expanding and occupying the pore spaces. With this continual reduction in porosity, it would be expected that larger tensile forces would be generated in the cement capillaries and a continual change in moisture would result in the clay sample over time. However, this is not evident from the moisture results recorded in the clay sample after 7 days. As mentioned, it is likely that the lack of moisture change after this time is due to the perspex tube reducing the external surface area available to the pore water causing the system to effectively shut itself off, once a certain degree of interaction has taken place between the cement and pore water.

The difference in the total porosity values recorded between the two methods is because MIP (pore diameter 3nm – 14 μm) covers a wider range of pore sizes in comparison to nitrogen adsorption (0.3 - 300 nm); with nitrogen results believed to be untrustworthy below diameter of 8nm (Aligizaki, 2006). This has been well documented in previous research, as has the bottleneck theory and the reliability of the testing procedures. However, the Author incorporated these tests into the test programme in order to enhance the findings and explanations as to what is happening within the cement as a result of it continual interaction with the pore water from the soil.

Capillary Sorption Test

The purpose of the capillary sorption tests were to provide further evidence that water absorption into the cement was continually taking place with an increase in time. The tests were conducted on cement samples which had utilised the pore water available in normally consolidated clay samples for 1, 3, 7, 14 and 28 days. The tests were performed as described in section 3.7.3 with the results obtained provided in Table 6.3.

Table 6.3: Capillary Sorption test results

Stage of Curing, Days	Graph Gradients, $(g/mm^2/time^{0.5}) \times 10^{-6}$	
	Control Sample, w/c ratio 0.5	Cement Inclusion
1	394	973
3	273	780
7	156	273
14	120	192
28	111	154

From these results, it can be shown that the sorptivity of the cement samples in contact with kaolin, have a higher tendency to absorb water than the control samples cured in a non-limiting environment. This was to be expected considering the different conditions placed on each sample during 'curing'. The results emphasise that the cement continues to absorb water with an increase in time, however at a constantly reducing rate as a result of the cements microstructural development in the presence of water. This development causes capillary pore sizes to reduce and an increase in the tensile force is generated in the capillaries; hence the continual absorption of water into the cement.

An absorption test was also performed on dry unhydrated cement grains passing a 600μm sieve. This test was performed in order to show the potential for water absorption when the dry cement is initially placed in contact with a clay soil. The sorptivity was found to be $4407 \times 10^{-6} g/mm^2/time^{0.5}$ for initially dry cement which is high in comparison to all hardened cement samples and confirms that the potential for dry cement to absorb water is extremely high.

6.3 INTERACTION BETWEEN AN OVERCONSOLIDATED CLAY SOIL AND A DRY CEMENT INCLUSION

6.3.1 Visual Inspection Performed During Testing

Lightly overconsolidated clay soils of OCR 1.5 and 2.0 were investigated after 7 days of interaction with an initially dry cement layer, in order to analyse whether the potential for cement to utilise the pore water from the soil is affected by the current stress state of the soil in relation to a preconsolidation stress of 140kPa.

Moisture tests carried out on control samples provided average moisture content results of 58% for the clay of OCR of 1.5 and 61% for OCR 2.0. Taking these values into consideration, coupled with the fact that the effective stress is known for each of the levels of consolidation, it was possible to plot the voids ratio against the effective stress (Figure 6.7). Where voids ratio (e) was found using the equation:

$$e = w \times G_s$$

Where: G_s = 2.69 for kaolin and w = moisture content.

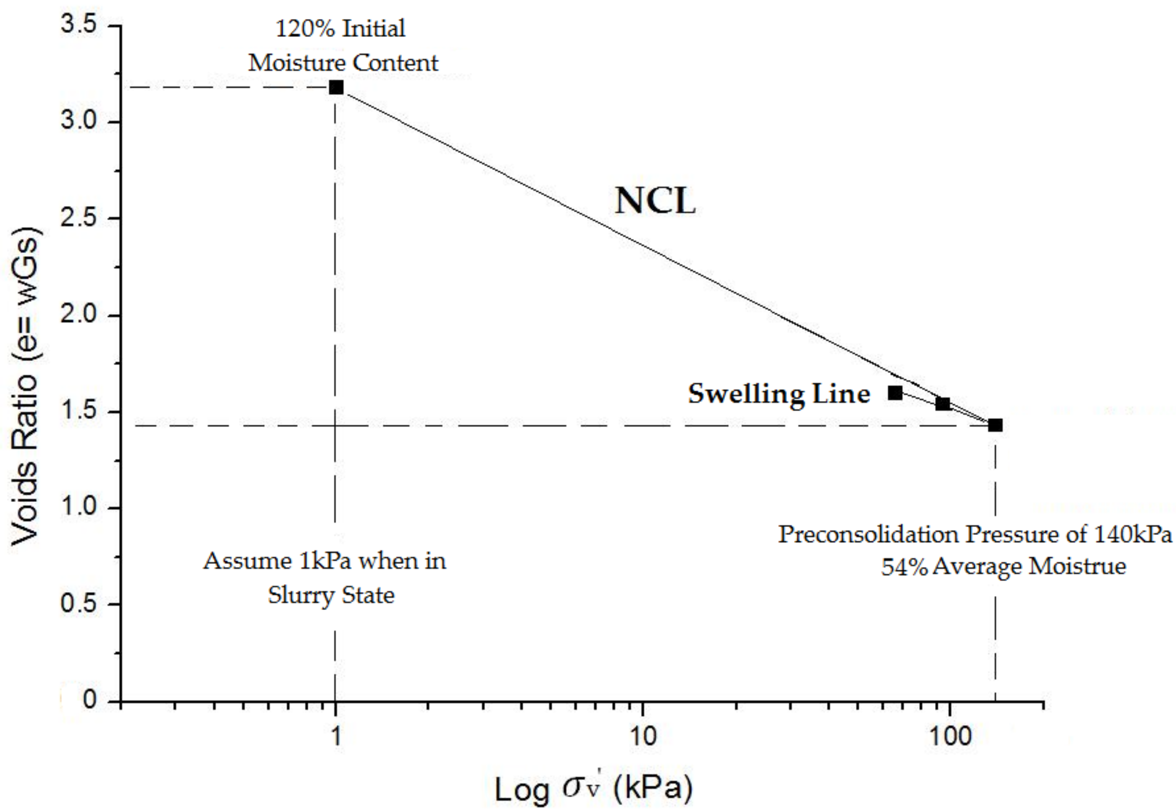


Figure 6.7: Voids ratio in relation to effective stress for each level of consolidation

As discussed in section 3.6.2, samples were initially mixed to 120% moisture content prior to consolidation, which provided a slurry mix well in excess of the liquid limit. For the purpose of Figure 6.7 it was assumed that the effective stress is equal to 1 kPa in this slurry state.

It has been well established that overconsolidated soils are denser and drier than normally consolidated soils (as can be shown in Figure 6.7). This is true for any particular effective stress, with higher values of OCR providing stronger and less compressible samples. As a result, it was anticipated prior to proceedings that the level of water absorption by the cement layer, hence the relative level of soil strength improvement, would reduce with an increasing value of OCR. This was somewhat evident by way of a visual inspection (refer to Figures 6.9 & 6.10), as the ability of the cement to draw water from the soil was seen to reduce with an increase in the OCR; as the rate at which water penetrated through the cement layer reduced.

Again, the initial water uptake in terms of depth of penetration through the cement layer was plotted against the square root of time, as shown in Figure 6.8.

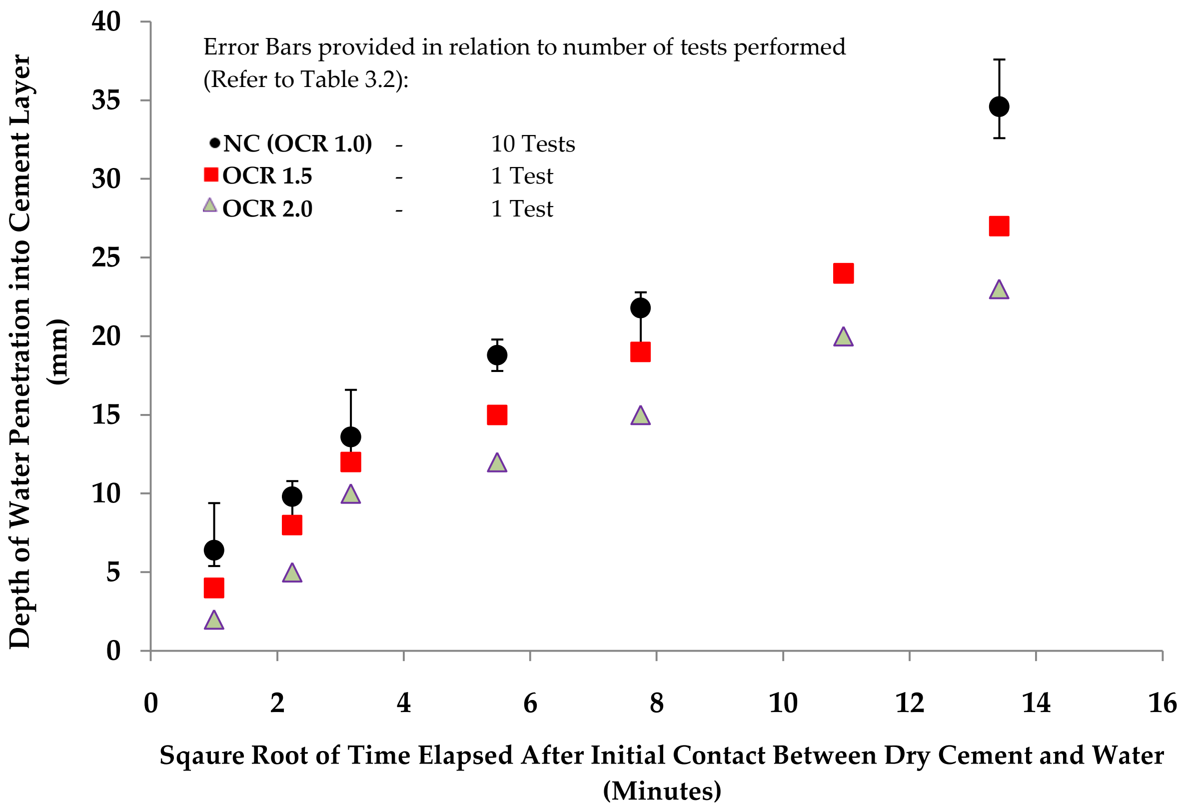


Figure 6.8: Water penetration through cement layer over time when interacting with soils of OCR = 1.0, 1.5 and 2.0

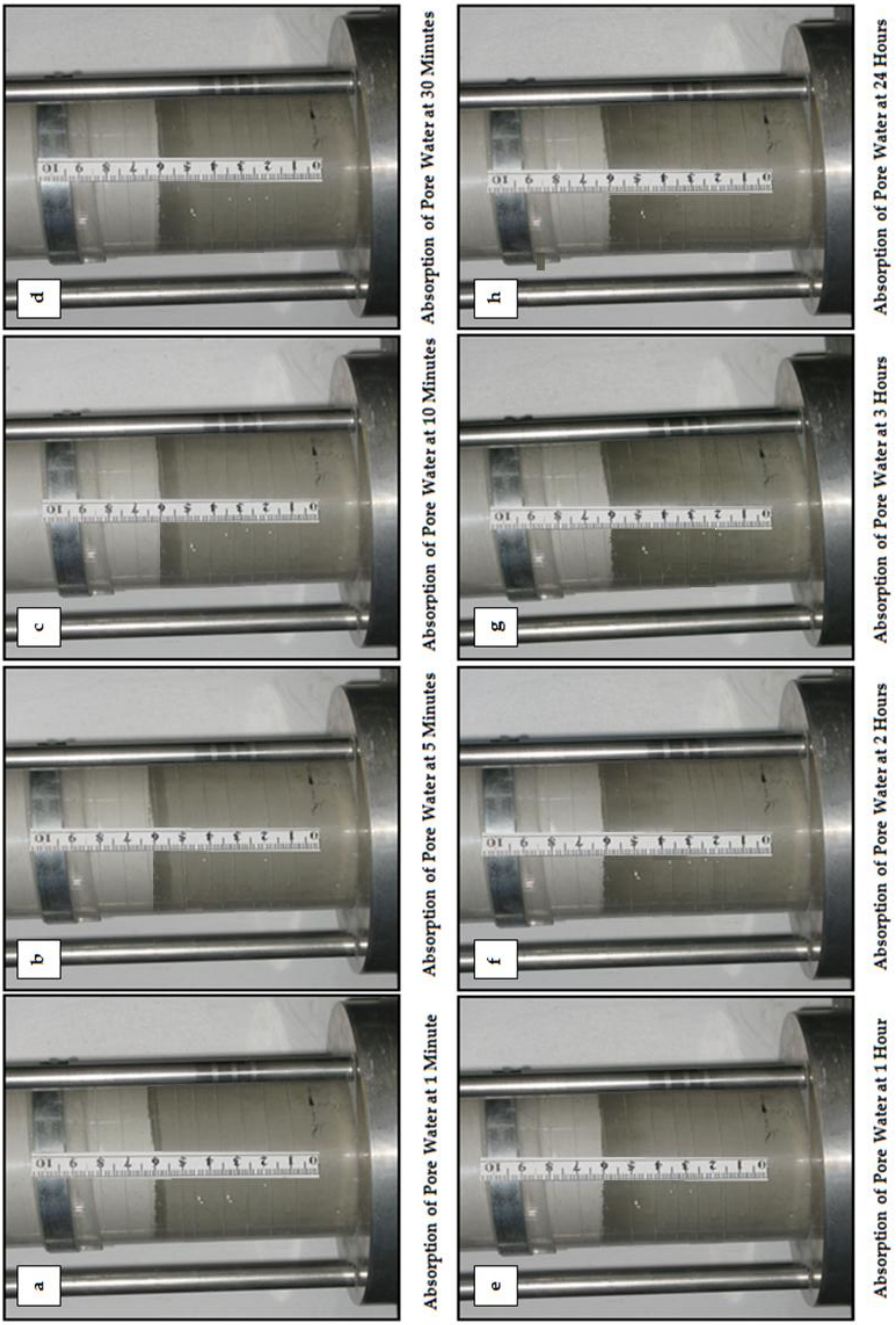


Figure 6.9: Visual representation of pore water absorption from a kaolin sample (OCR = 1.5) into cement layer

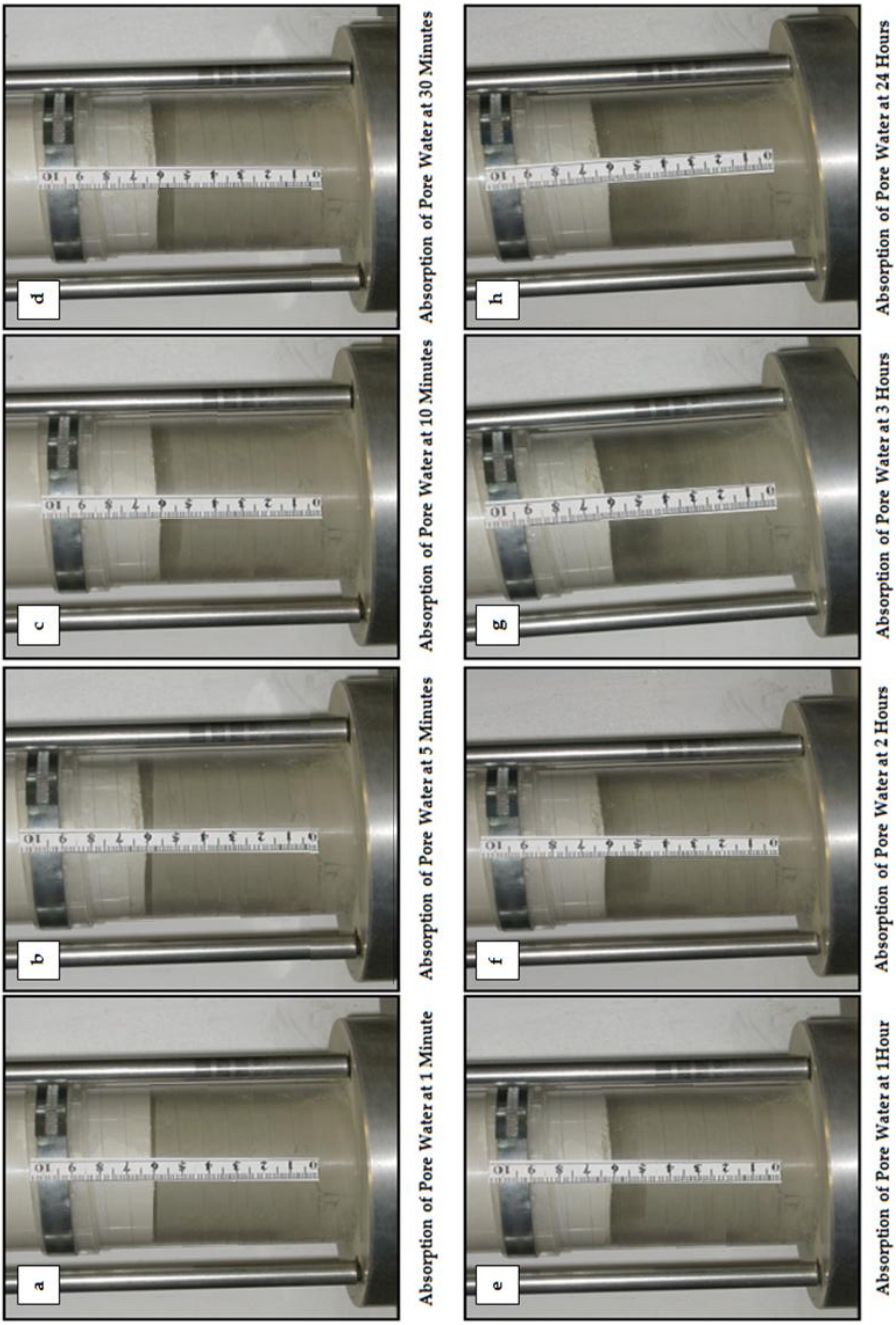


Figure 6.10: Visual representation of pore water absorption from a kaolin sample (OCR = 2.0) into cement layer

By taking the slope of each of the plots from Figure 6.8, it was found that the general rate of water penetration through a cement layer reduces with an increase in OCR; with clay of OCR 1.0 providing a rate of 2.1mm/minute$^{0.5}$, OCR of 1.5 generally 1.8 mm/minute$^{0.5}$ and clay with OCR of 2.0 generally having a rate of 1.6 mm/minute$^{0.5}$. As a result of these reduced absorption rates with increasing OCR value, it was anticipated that the volume of moisture change experienced in the soils would also reduce.

6.3.2 Kaolin Clay Soil Testing

The moisture content results extracted from soils of different OCR can be viewed in Figure 6.11.

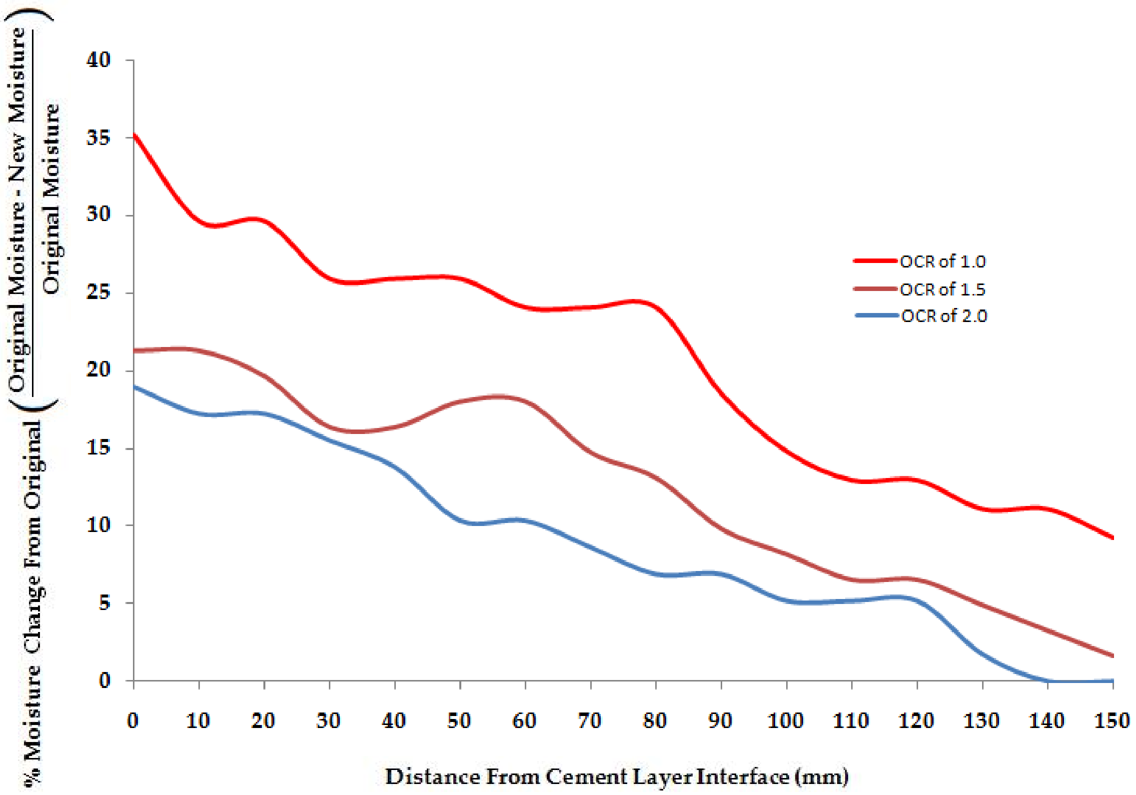


Figure 6.11: Moisture movement recorded for clay soils of OCR = 1.0, 1.5 and 2.0 after 7 days contact with cement layer

As predicted, the results from Figure 6.11 indicate that the cement layer does not provide the same level of moisture reduction within overconsolidated clay as it does when in a normally consolidated state; with higher moisture changes recorded along the length of the clay sample for lower values of OCR. This is to be expected, as lower values of OCR move the soil closer to the NCL where the soil is softer and much more likely to experience a greater change in moisture. The trend in results also suggests that the distance to which moisture change occurs, away from the cement-soil interface, reduces with an increase in OCR.

Hypothetically, using the shear strength relationship with liquidity index (refer to Figure 4.1) it can be calculated that that the moisture reduction recorded at the interface for a soil with OCR 1.0 relates to a strength improvement of 696% in comparison to the clay in an untreated condition. This eclipses the strength improvement of 313% at the interface for a soil of OCR 1.5 and 232% for OCR 2.0.

6.3.3 Hardened Cement Inclusion Analysis

After extracting the hardened cement from the perspex tube, a visual inspection was performed on the samples. From these inspections (Figure 6.12), it was likely that that the degree of hydration would be greater at the cement-soil interface for both samples, as darker shades of grey were visible where the cement was in direct contact with the soil (darker areas indicate a greater reaction with water).

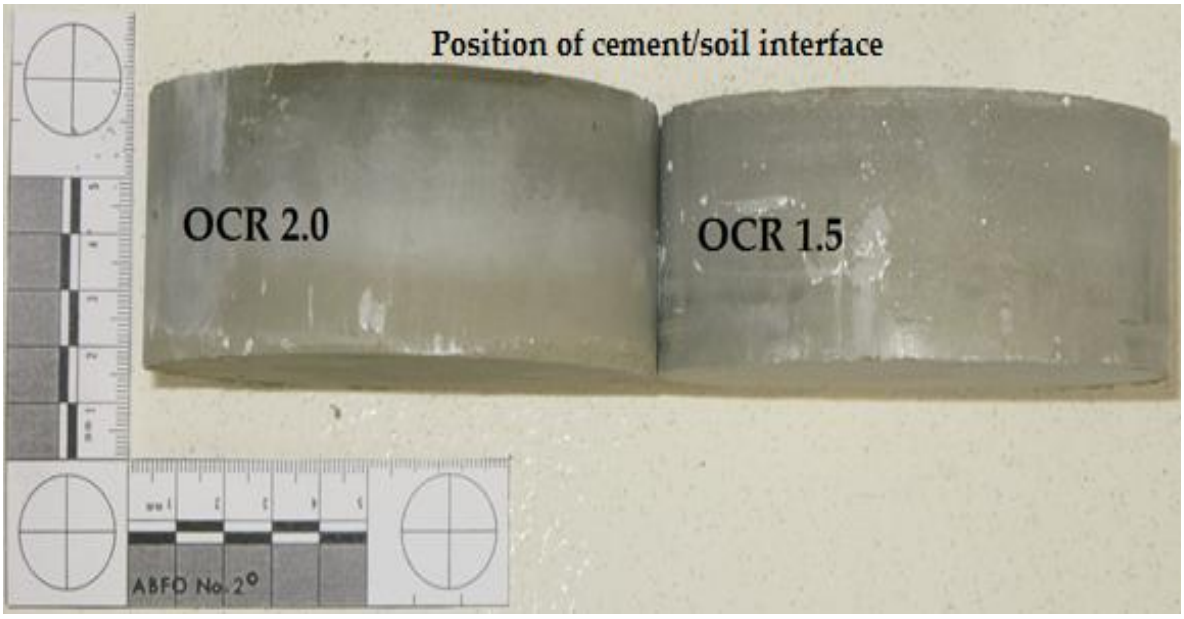


Figure 6.12: Colour profile through hardened cement samples after 7 days in contact with clay soils (OCR = 1.5 and 2.0)

The samples were also weighed upon extraction and revealed that 41% of water, by weight of dry cement, was absorbed by the cement layer in contact with clay of OCR 1.5 and 37% absorption for clay of OCR 2.0. This is reduced in comparison to the 48% absorbed by a normally consolidated soil at 7 days. From this information, it was also likely that the degree of hydration would be greater for cement in contact with soils of higher OCR. Again, samples for TGA testing were extracted at 10mm depths through the cement layer to generate a water penetration profile, based on the degree of hydration of the cement. The results are expressed graphically in Figure 6.13.

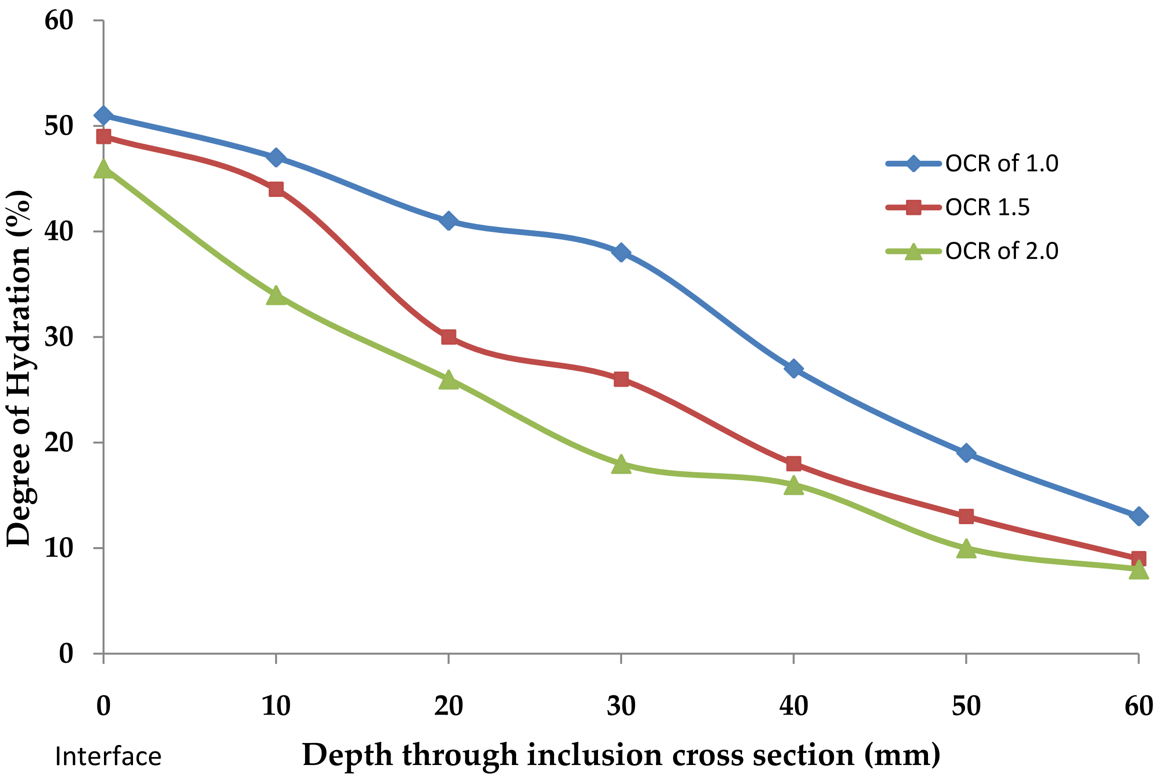


Figure 6.13: Hydration profile through cement cross section after 7 days in contact with clay soils of OCR 1.0, 1.5 and 2.0

As predicted, the results indicate that the degree of hydration is greatest at the interface, with the cements interaction with water shown to reduce with increasing depth into the cement layer. The OCR value can be shown to have an influence on the level of hydration as an increase in OCR results in a lower degree of hydration. This will be the result of an increase in the stiffness of the clay as this limits the amount of pore water available to the cement.

6.4 INCLUSION PERFORMANCE IN THE EVENT OF A REDUCTION IN CONFINING STRESS

There are a number of potential problems that could be encountered when using this proposed method of stabilisation on site, which are hard to predict at this current time without further research. However, for this section of the Chapter a worst case scenario involving a reduction in confining stress is proposed. This scenario could potentially be encountered on site during the installation process; as the removal of soil to accommodate the cement inclusion could cause stresses in the surrounding soil to relax and as a result could limit the performance of the cement inclusion in stabilising the surrounding soil.

Two tests were conducted in the 1-D perspex compression rig on kaolin clay initially consolidated to 140kPa, in order to investigate the performance of a cement layer in the event of this worst case scenario occurring. The tests were performed in such a manner that a reduction in overburden pressure within the perspex tube was used to represent a reduction in confining stress in practice. Unlike the overconsolidated samples (which are unloaded and swell with full access to water until equilibrium is achieved), the samples used to represent a reduction in confining pressure were denied access to water during unloading. Pressures of 94kPa (1/4 the preconsolidation stress) and 70kPa (1/2 the preconsolidation stress) were arbitrarily selected prior to testing. The cement layer remained in contact with the clay for a period of 7 days; which again was an arbitrary number, with moisture content samples being extracted from the clay sample and TGA being performed on the hardened cement.

6.4.1 Kaolin Clay Soil Testing

Moisture content samples were extracted from the clay in the same manner as for all previous tests i.e. 10x10x10mm sample sizes taken at increasing distances from the cement-soil interface. The results can be viewed in Figure 6.14.

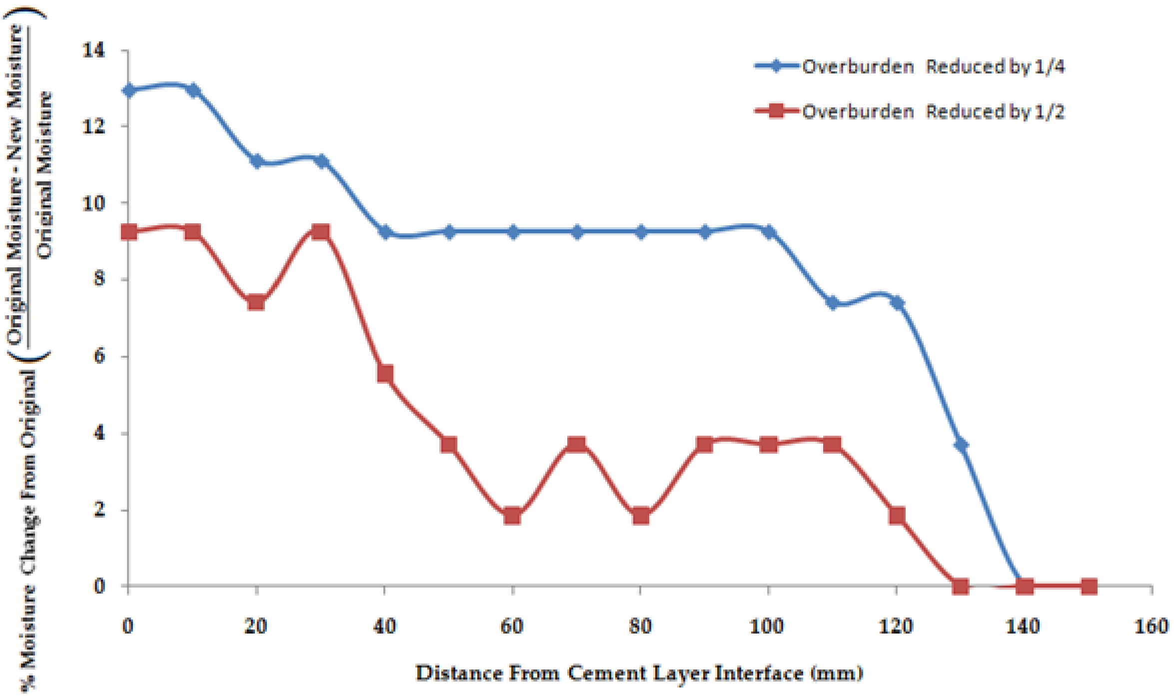


Figure 6.14: Moisture change recorded throughout clay after reduction in overburden stress and 7 days contact with cement layer

From these results, it appears that the level and distance to which moisture change is experienced reduces with a reduction in overburden stress. Hypothetically, using the liquidity index and shear strength relationship, the strength improvement at the interface in comparison to an untreated clay is 115% if the overburden pressure is reduced by ¼ and 73% in the event that the overburden stress reduces by a ½, which suggests that the relative improvement in the soil reduces with a reduction in overburden stress.

It is suggested that as a result of the reduced stress, the soil attempts to swell in the absence of water leading to the generation of suction forces within the clay matrix. As the cement comes into contact with the clay in this condition, the attractive forces which are required to initially draw water into the cement layer must first overcome these suction forces. This limits the ability of the cement to absorb water and as a result reduces the moisture change, hence relative improvement, which takes place in the surrounding clay soil.

The limited ability for water to penetrate through the cement layer with a reduction in overburden stress was confirmed in the TGA results (Figure 6.15).

6.4.2 Hardened Cement Inclusion Analysis

TGA tests were again performed on cement samples extracted at 10mm intervals from the cement-soil interface, with the results presented in Figure 6.15.

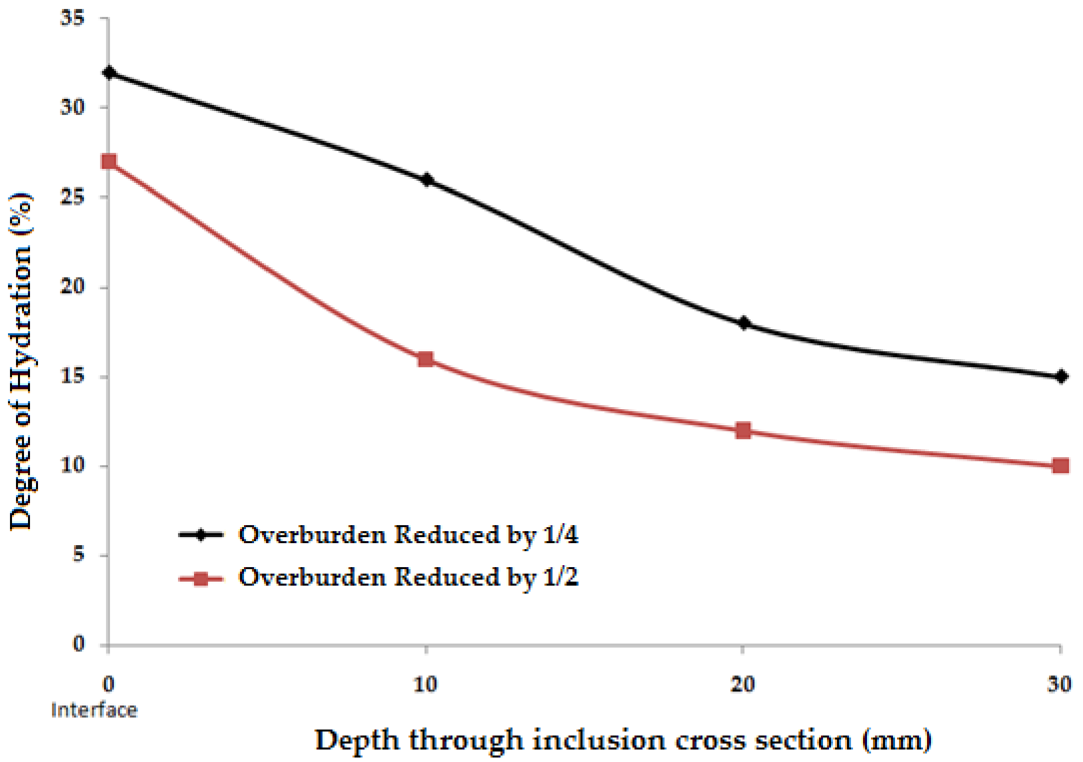


Figure 6.15: Hydration profile of cement layer after 7 days in contact with a clay soil experiencing a reduction in overburden stress

The results recorded in Figure 6.15, confirms that there is a reduction in the level of water absorption into the cement layer with reducing overburden stress, again with the degree of hydration shown to reduce at increasing depths from the cement-soil interface.

Unfortunately for both tests, cement at distances greater than 30mm from the cement-soil interface could not be accurately tested to maintain a hydration profile, due to the cement crumbling away during extraction. It is believed that the cements limited interaction with water at these distances led to the cement not hardening sufficiently; as the colour and texture of the cement at distances greater than the 30mm from the interface resembled the loose unhydrated cement used to form the layer at the beginning of testing. For this same reason the weight of water absorbed by the cement layer could not be accurately determined; due to a loss of material during the extraction process.

The inability of water to penetrate and react with the cement at a distance greater than 30mm from the interface has also been accredited to suction forces being generated within the clay soil matrix. As mentioned, the cement would have to overcome the suction forces in the soil before any attractive force would draw pore water in to the cement. Thereby, less water is drawn into the cement and the depth to which water penetrates the sample reduces.

From the results recorded in both the clay and hardened cement, it can be shown that a continual reduction in confining stress (represented in this case as a reduction in overburden stress) can adversely affect the performance of the cement, with respect to moisture changes experienced in the surrounding soil. It is suggested that the cements ability to absorb pore water from a clay soil reduces with a reduction in confining stress. This is the result of suction forces being generated in the clay matrix, which must be overcome before any absorption by the cement can take place. As an increase in suction force can be expected with continual reduction in confining stress, it can also be expected that the ability for the cement to absorb water will reduce; resulting in reduced moisture changes and a reduced level of strength improvement to the surrounding soil.

6.5 CHAPTER SUMMARY

From the results and observations recorded in this section of work the following conclusions can be drawn by the Author:

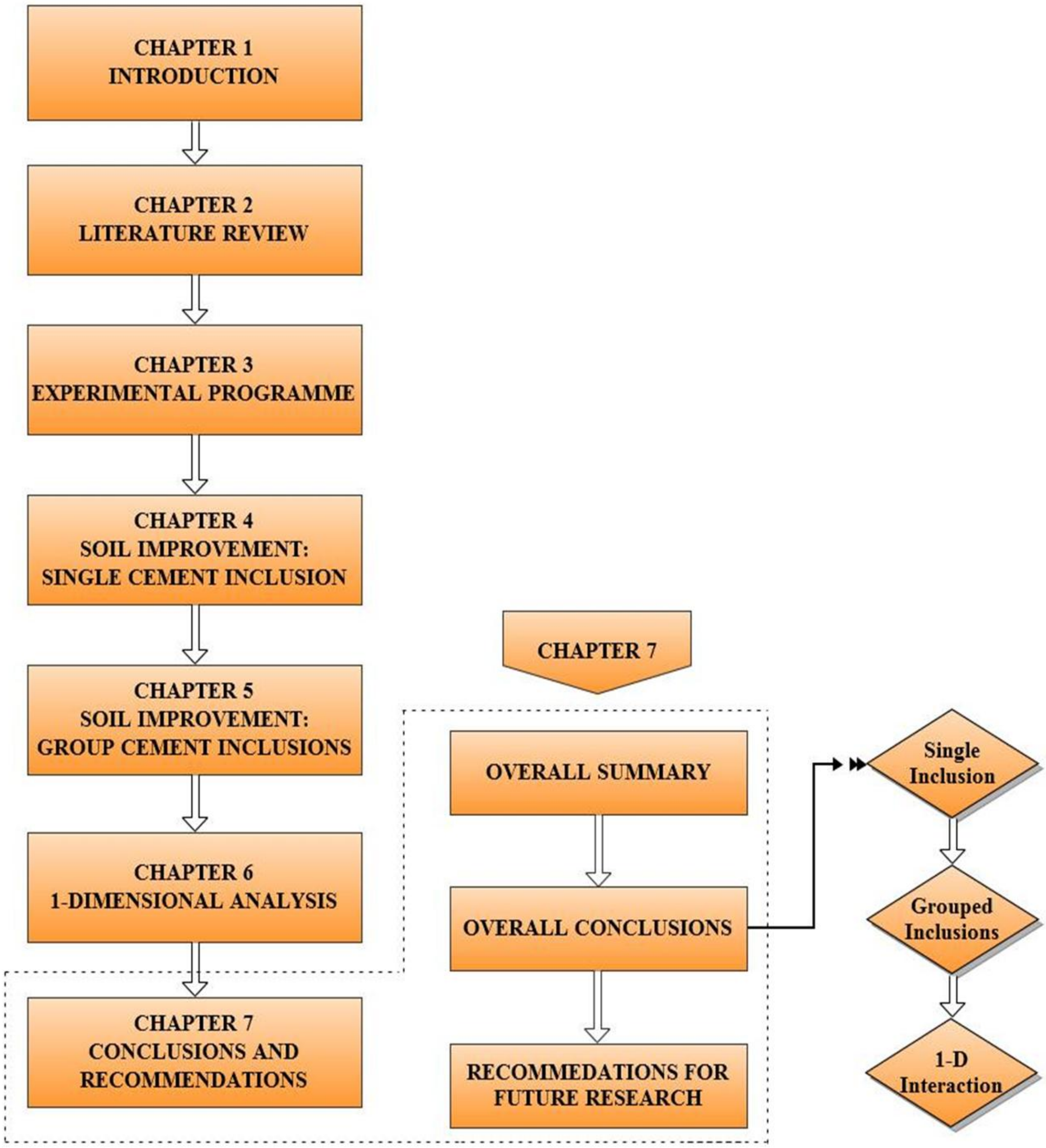
- Three mechanisms are accredited to cement utilising pore water from the surrounding clay; absorption, diffusion and capillary sorption. Absorption and diffusion are responsible for the water penetrating into the cement layer prior to cement hardening, with capillary sorption allowing water to continually penetrate and react with the cement at increasing depths from the interface once hardening has taken place.

- Time is shown to have an influence on the development of the cement; with the pore water from the soil shown to reach greater penetration depths with an increase in time. However, with respect to moisture changes in the clay an increase in time only influences the system up to 7 days, at which stage the system appears to shut down. This is not consistent with the behaviour of the clay in Chapter 4; where an increase in contact time with the cement layer, was shown to provide continued moisture movements. The limited surface area available to the pore water within the perspex tube is believed to be the cause of the difference in behaviour.

- Dry cement is capable of utilising pore water from both normal and overconsolidated clay soils in order to initiate a hydration reaction. However, the volume of pore water utilised from the clay reduces with increasing value of OCR. As a consequence, the scale of improvement recorded in the soils strength is also reduced.

- The rate at which water penetrates through the sample is dependent on the OCR value of the clay, with OCR 1.0 penetrating the cement at a rate of 2.1 mm/minute$^{0.5}$, clay of OCR 1.5 at a rate of 1.8 mm/minute$^{0.5}$ and 1.6 mm/minute$^{0.5}$ for clay of OCR 2.0.

- A reduction in confining stress can be shown to inhibit the ability of the cement to utilise pore water from the surrounding clay soil; with reduced moisture changes being recorded for reduced confining stresses. The depth to which water penetrates the cement layer was also shown to reduce with a reduction in confining stress. This has been proposed as a worst case scenario, as the relaxation of stresses in the surrounding soil can occur during installation as a result of the removal of soil to facilitate the cement inclusion.



"There is no such thing as a failed experiment, only experiments with unexpected outcomes"

Richard Buckminister Fuller (1895-1983)

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 OVERALL CONCLUSIONS

This Chapter presents the overall conclusions drawn from the different test series undertaken during this study with respect to the aim and objectives defined in Chapter 1. Recommendations have been made which highlight areas which require further research and consideration.

The overall aim of this study was to prove; using an experimental approach, whether the placement of dry unhydrated cement had the ability to enhance the characteristics of the surrounding clay soil, in particular strength via consolidation. This study was needed to provide evidence that the use of dry Portland cement could utilise pore water from a clay soil in order to hydrate, without the need for mechanical mixing. Additionally soil dehydration, as a result of moisture transport into the cement inclusion, can enhance the strength properties of the surrounding soil, and potentially enhance the level of improvement obtained from the lateral soil displacement performed in CMC technology. To achieve this aim, a laboratory scaled experimental programme involving cement inclusions in pure kaolin clay, at different moisture contents, was carried out and a series of objectives were devised to ensure successful delivery of the aim. The objectives and the extent to which they were successfully achieved are reviewed in Table 7.1.

Each clay sample was mixed by hand, to moisture contents of 40% (shear strength 50kN/m^2) and 60% (shear strength 6kN/m^2), using the exact same mixing process throughout. This allowed an analysis of the dry inclusions performance in clay samples consisting of different physical properties to be performed. Ten inclusions consisting of Portland cement and diameters of 38mm and 70mm respectively, were installed into the clay soil of 40% moisture for 1, 3, 7, 14 and 28 days. Additionally, one 100mm Portland cement inclusion tested after 28 days contact with clay of 40% moisture and one 38mm CSA inclusion tested at 1 day were included in the test programme. Five 38mm Portland cement inclusion samples were investigated at 60% clay moisture.

Table 7.1: Summary of research objectives and the extent to which they were achieved

OBJECTIVE	ACHIEVED?	TO WHAT EXTENT WERE OBJECTIVES MET?
Review established stabilisation techniques that involve the use of dry cement to utilise pore water from the soil and explore the theory behind their ability to improve the soil.	✓	Reviewed large volume of literature with respect to established soil stabilisation techniques such as lime-cement columns, stone columns and CMCs. Briefly described the genesis and theory behind each technique and their abilities to strengthen soil. Review of cement hydration and soil strength improvement through reduction in moisture content also performed.
Devise scale experimental test methodology and carry out laboratory tests to determine if soil improvement can be achieved using a dry cement inclusion.	✓	Executed a full range of laboratory scaled experiments focusing on the ability of the dry Portland cement to utilise soils pore water and initiate hydration. Strength improvement of soil tested over a range of test times up to and including 28 days.
Determine whether cement hydrates to the inclusion core as a result of its interaction with pore water in the soil.	✓	Thermogravimetric Analysis (TGA) allowed a profile of water penetration through the cement inclusion to the central core to be generated, with respect to time.
Examine the radial influence of a dry cement inclusion and its ability to enhance the soils strength.	✓	Radial influence of inclusions measured through changes to moisture content at increasing distances from inclusion interface.
Examine the influence of the inclusion diameter on radial improvement in the soil.	✓	Compared the radial influence of 38mm, 70mm and 100mm inclusions at 28 days through monitoring moisture changes in a soil of 40% initial moisture content.
Design a one-dimensional rig capable of providing both physical and visual inspections to be carried out with respect to clay soils interaction with a dry cement inclusion, and perform tests in the rig focusing on a range of soil loading conditions.	✓	One-dimensional rig designed, manufactured and fully operational, with both physical and visual data possible. Rig calibrated with pneumatic pressures set to some value x to allow a known stress to be applied to the sample, in relation to a preconsolidated stress (i.e. range of OCR conditions can be tested).
Develop an empirical description to explain the general effect of soil dehydration through cement hydration.	✓	Determined improvement in soil strength through liquidity index relationship with shear strength as presented in Atkinson (2007).
Assess the performance and environmental benefits of incorporating a different material in place of Portland cement to form the inclusion.	✓	Single test performed incorporating Calcium Sulfoaluminate (CSA) cement in place of traditional Portland cement (PC). Test performed at 1 day with 38mm inclusion diameter investigated. Literature review performed on CSA.

The dehydration of the surrounding soil was monitored by extracting moisture content samples at increasing radial distances and depths from the inclusion interface. Each clay mix consisted of a control sample in order to confirm that dehydration of the soils moisture was a direct result of the dry inclusion utilising pore water. Thermogravimetric Analysis (TGA) analysis was performed on each of the hardened cement inclusions, with the degree of hydration providing an indication of water movement through the inclusion over time. Both mechanical and hand shear vanes were used to confirm that the soils strength had improved as a result of the dry cement inclusions introduction.

The performance of grouped dry cement inclusions was also investigated, with the three group arrangements consisting of square, triangular and circular grids being based on the literature provided for both CMC and deep soil mixing techniques. These tests were all performed at 28 days in clay soil of 40% moisture content, again with moisture extractions and hand shear vanes adopted for investigating soil improvement.

A further study focused on the 1-dimensional interaction between the dry cement and the pore water in the clay soil. This study provided both visual and physical data and incorporated kaolin clay subjected to different stress conditions. Two sets of tests, involving normally consolidated kaolin (preconsolidation stress of 140kPa), were performed after 1, 3, 7, 14 and 28 days contact between the cement and the clay soil. Moisture extractions were taken from the clay at each of these times and for each set of tests. The first set of cement samples were tested using TGA, Mercury Intrusion Porosimetry (MIP) and Nitrogen Adsorption in order to monitor the movement of water through the cement layer. The second set of cement samples were tested for capillary sorption in accordance with ASTM C1585-04.

As the behaviour of a soil does not always exhibit that of a normally consolidated soil, it was necessary to investigated whether the ability of a cement inclusion to utilise moisture from a clay soil was dependent on its current stress state in relation to a preconsolidation stress of 140kPa (i.e. an overconsolidated state). Two tests involving OCR values of 1.5 and 2.0 were included in the test programme, with each inclusion permitted to interact with the soils pore water for a period of 7 days. TGA was again performed on the cement layer, with moisture samples extracted from the kaolin at increasing radial distances from the cement interface.

A final set of 1-dimensional tests were performed in order to assess the performance of the dry cement inclusions in the event of a worst case scenario; proposed to be a reduction in confining stress from the surrounding soil.

7.1.1 Performance of a Single Cement Inclusion

The primary question was whether or not the cement would hydrate to the core of the inclusion without the mechanical mixing process, argued to be essential in the deep mixing methods, and whether the transport of water through the inclusion would result in a radial dehydration of the surrounding clay soil, thus increasing the strength.

The two soils investigated in this study provided evidence that this technique is suitable for only certain clay soil conditions. For soils consisting of 60% moisture (initial strength of 6kN/m^2), the installation of a dry cement inclusion can be observed to have a detrimental influence on the soil strength; particularly in a zone immediately surrounding the inclusion-soil interface. This is the result of a pressure gradient in the soil causing water to migrate towards the inclusion at a much faster rate than the rate at which water is absorbed into the cement inclusion (particularly true after cement hardening), leading to a build up of pore water in this vicinity of the inclusion. On the other hand the installation of a dry cement inclusion can be seen to have favourable effects on the strength properties of kaolin clay soil mixed to 40% moisture. Again a pressure gradient, caused by the immediate absorbtion of pore water surrounding the inclusion, entices pore water at increasing radial distances to migrate towards the inclusion. However, in this case the ability of the inclusion to absorb the water is at a faster rate than the migration of water to the inclusion, thereby causes a reduction in moisture content.

In both soil conditions, the time can be shown to have a significant influence on the moisture movement through the soil. An increase in time has a beneficial influence on clay of 40% moisture at all locations in the sample, with moisture reductions shown to occur at increasing radial distances from inclusion interface. The behaviour of the 60%

clay soil is shown to act differently with increased time. At increasing radial distances from the inclusion interface, the soil shows an increase in strength which is the result of moisture moving towards the inclusion under a pressure gradient. However, as a result of this moisture movement and the continually decreasing rate at which water enters the inclusion, the moisture content can be shown to increase in regions surrounding the inclusion interface. Thereby, both improvement and decline in soil shear strength is observed in this sample.

The influence of the inclusion diameter has also been shown to have a significant influence on both the magnitude of improvement in strength and the radial distance to which improvement is experienced. This is a direct result of an increase in cement content, as more water would is required to be absorbed in order to satisfy the cements natural affinity for water. TGA results recorded for both the 38mm and 70mm inclusions indicate that water has access to the core of the inclusion, as the degree of hydration is continually shown to increase with an increase in time. As expected the time for water to penetrate and react with the core increases with increasing diameter; with TGA results for the 100mm inclusion indicating that water had managed to penetrate to the inclusion core, however not to the same degree as the cement at the interface as was evident in both 38mm and 70mm inclusions.

The material used to form the dry cement inclusion can also have an effect on the behaviour of the soil. CSA shows a larger increase in the magnitude of soil shear strength improvement in comparison to the Portland cement inclusion. As CSA has more environmental benefits in comparison to PC, this material can be said to be a suitable replacement for PC up to a time of 24 hours after installation. However, without tests involving longer times it is not possible to make any other comments regarding CSA and its benefits to soil improvement over a long term basis. This is out with the scope of this study.

7.1.2 Performance of Grouped Cement Inclusions

With the local dehydration effect established for single cement inclusions in kaolin clay of 40% moisture, the ability of grouped inclusions to interact and enhance the strength properties of the soil at this moisture was investigated. Within the findings from this study, all proposed group arrays displayed a greater degree of improvement in comparison to an isolated inclusion at 28 days, with improvement identified to be dependent on the following factors; area replacement ratio, group arrangement and inclusion spacing.

The area replacement ratio was shown to have an influence on the magnitude of strength improvement; with increased strengths recorded for increased area ratios. This was expected prior to testing as the introduction of more (or larger) inclusions ultimately leads to a greater deal of water being effectively 'removed' from the soil in order to satisfy the increased cement contents demands.

However, the arrangement of inclusions and their spacing relative to one another appears to have the most significant influence; with a triangular grid arrangement found to be the most effective in terms of recording the largest improvements in strength. This was most likely the result of the close proximity of the inclusions, in relation to one another, allowing the local dehydration effect experienced with singular inclusions to 'overlap' and generate an area effect. The closer the inclusions, the greater attractive forces experienced by the pore water towards the inclusions and the smaller the distance for the pore water to travel; resulting in further reductions in the soils moisture. This can be said to be true as the triangular arrangement (which provided the highest improvement) had the same area replacement ratio as that of a circular arrangement adopted in section 5.5, however with closer knit inclusion spacings. The results from the initial spacing tests performed in Chapter 5 further enhance this reasoning.

Increasing the inclusion diameter was shown to allow larger spacings between inclusions to be adopted, whilst maintaining a large degree of strength improvement in comparison to that achieved by a single inclusion. This was the result of larger area effects being generated by an increase in the inclusion diameter and an increase in the cement content.

Tests performed on the hardened cement inclusions identified that between 50-54% water, by weight of dry cement, was absorbed by each of the 38mm inclusion, with the core of the inclusion provided with continual access to water in all but one of the nine inclusions tested. These results were echoed in the 70mm inclusions; however the volume of water absorbed by the 70mm inclusions was larger with 55-61% water absorption recorded. This again is the result of an increase in cement content removing a larger portion of water from the soil.

7.1.3 1-Dimensional Interaction between Dry Cement Inclusion and Pore Water

By way of a visual inspection, the ability of a dry cement inclusion to utilise pore water from a clay soil can be accredited to three phenomenon; initial adsorption, diffusion and capillary sorption. As contact between the cement at the interface of the inclusion and the kaolin clay occurs, an immediate adsorption of water is performed by the cement in order to satisfy its natural affinity for water. This adsorption creates a concentration difference between the interface and the dry unhydrated cement within the sample, causing water to travel through the sample by the process of diffusion. After a period of approximately 3 hours the ability of the cement inclusion to absorb water is reduced; as indicated by a decline in the rate at which water travels through the dry cement, and the diffusion mechanism; by which water was travelling through the inclusion, changes to capillary sorption. This is the result of the cement at the interface hardening causing a permeable seal to form which inhibits water penetration through the inclusion.

Similarly to the results noted in 7.1.1 the time has a significant influence on the strength improvement recorded in the surrounding soil, with an increase in moisture reduction recorded with increased time.

The OCR value is also an influential factor, as an increase in OCR value causes a decrease to the scale of moisture reduction experienced in the surrounding soil. This is the result of increasingly overconsolidated soil being stiffer and drier than soils of lower OCR, thereby reducing the ability of the dry cement to utilise pore water from the soil.

The performance of the system was also investigated in the event of a worst case scenario, which as mentioned is proposed to be a reduction in confining stress provided from the surrounding soil after the installation of the inclusions. From this set of tests, it was concluded that the ability of the dry cement to utilise the soils pore water, hence cause the radial dehydration effect necessary to improve the soil strength, reduces with increasing reduction in confining stress. The ability of water to penetrate through the cement was also reduced. Therefore, in situations where a reduction in confining stress is likely to be encountered to a large extent, the adoption of dry cement inclusions to stabilise the soil would potentially be avoided. However, further research should be undertaken to fully understand the limitations of this technique with confining stress.

This research has achieved its original aim and has shown that there is significant potential for dry cement inclusions to be used for stabilising clay soil; given the right soil conditions. However, it must be stressed that this research was performed under controlled laboratory conditions, with homogenous clay samples of consistent moisture content. In the field, it is likely that a number of soil conditions and problems not considered within this research could be encountered which could potentially limit the effectiveness of this technique in strengthening the surrounding soil. The following recommendations for future research will further enhance the understanding and limitations to this proposed method and should be investigated thoroughly before adoption of this technique on site is considered for soil stabilisation.

7.2 RECOMMENDATIONS FOR FUTURE REASEARCH

i. This study generated a novel approach to consolidating clay samples of different moisture content, with the introduction of dry cement inclusions dehydrating the soil in order to hydrate. This study focused on the performance of these inclusions in homogenous laboratory prepared pure kaolin. However, it would be of interest for field tests to take place involving a range of *in situ* soil conditions such as clays of different moisture content or layers of different material. This would allow a more realistic analysis, or identification of limitations, to the proposed method to take place before being considered for use in practice.

ii. The material used to form the dry cement inclusions studied in this research work consisted of 100% Portland cement, with a single test involving CSA as a replacement. It would be of great benefit for a range of different materials and mix combinations to be incorporated into the inclusion, in order to determine the influence of these materials on the strength improvement of the surrounding soil.

Incorporating such materials as Pulverised Fly Ash or CSA is one suggestion, as these materials have the potential to either slowly or rapidly dehydrate the soil. The use of binary or ternary mix combinations involving Portland cement in different quantities is a further suggestion.

iii. An analysis involving grouped dry cement inclusions in a kaolin clay soil of 60% moisture would be useful to determine if an overall area improvement effect could be generated. From this study, the local effect of a single inclusion can be seen to decrease soil strength in a zone surrounding the inclusion with some form of improvement noted at increasing radial distance; particularly with increased time. A group test could determine if it is possible to obtain a central zone of improvement at centre of a inclusion arrangement, thereby providing some use for this technique in this soil condition.

iv. As this study of work focused on the performance of these inclusions up to 28 days, it would be of interest for curing times to be increased to possibly 3, 6, 12 or 24 months after installation. This would allow some durable aspects of this technique to be identified, i.e. the physical properties of the hardened inclusion at these stages could be examined to determine if deformation or deterioration has occurred.

v. Since the inclusion diameter in this study consisted of smaller scaled diameter in comparison to those produced from CMC or deep mixing techniques in practice, it is recommended that an investigation involving larger inclusion diameter be conducted over a range of time intervals. This would allow any practical limitations to the inclusion diameter to be identified, i.e. no water reaching the core of an inclusion of diameter x.

vi. An investigation involving different inclusion shapes is a possibility. This would allow a better understanding of the influence of surface area ratio on the rate of hydration to be known and allow the influence of the inclusion shape to be identified. Once a single inclusion shape has been tested a range of group patterns could then be tested.

vii. Investigating the ability of the dry cement inclusions to utilise pore water from both normally and overly consolidated soils in a 3-dimensional analysis would be useful. This could be conducted in field tests or in large consolidation equipment capable of providing the right conditions. Again a larger range of inclusion diameter and shapes could be investigated over a range of times in these soil conditions. It is also possible to further test the cements performance with increasing values of OCR. The soils tested in this study were lightly overconsolidated and it may be necessary for heavily overconsolidated clays to be investigated.

REFERENCES:

ACI Committee 302. (2006). Concrete moisture basics. In A. C. 302, *302.2R-06: Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (pp. 302.2R-12).
America Concrete Institute (ACI).

Åhnberg, H., Johansson, S.-E, Retelius, A., Ljungkrantz, C., Holmqvist, L. And Holm, G. (1995) Cement och kalk for djupstabilisering av jord – En kemisk fysikalisk studie av stabiliseringseffekter (Cement and lime for stabilisation of soil at depth – a chemical physical investigation of soil improvement effects). *Swedish Geotechnical Institute*, Linkoping, Sweden, Report No. 48, pp. 213.

Ambroise, P. (2004). New applications of calcium sulfoaluminate cement. *Cement and Concrete Research , 34* (9), pp. 671-676.

Arulrajah, A. (2009). Ground Improvement Technique for Railway Embankments. *Proceedings of the Institution of Civil Engineers , Ground Improvement 162* (G11), pp. 3-14.

Atkinson, J. (1987). Basics Mechanics of Soils. In F. G. Bell, *Ground Engineer's Reference Book* (pp. 12-25). London: Butterworth and Co Ltd.

Atkinson, J. (2007). Consolidation. In J. Atkinson, *The Mechanics of Soils and Foundations - Second Edition* (pp. 215 - 218). Oxon: Taylor & Francis.

Baker, R. W. (2004). Membrane Transport Theory. In R. W. Baker, *Membrane technology and applications* (2nd Edition, pp. 15-18). West Sussex: John Wiley & Sons Ltd.

Balfour Beatty. (2010). *Balford Beatty: Solutions: Ground Improvement: Vibro Stone Columns*. Retrieved June 17, 2010, from Balfour Beatty Web Site:
http://www.stent.co.uk/?page=VibroStoneColumns

Barclay (2009) - Private communications

Basheer, L. (2001). Assessment of the durability of concrete from its permeation properties: a review. *Construction and Building Materials: An international journal dedicated to the investigation and innovative use of materials in construction and repair, 15* (2-3), pp. 93-103.

Black, L. “Hydration of tricalcium aluminate (C3A) in the presence and absence of gypsum—studied by Raman spectroscopy and X-ray diffraction.” *Journal of Materials Chemistry* 16 (January 2006), pp. 1263-1272.

Brown, M. J. (2010) - Private communications

Bulut, R. (2001). *Total and Matric Suction Measurements with the Filter Paper Method.* Retrieved April 5, 2010, from Total and Matric Suction Measurements with the Filter Paper Method:
http://www.foundationperformance.org/pastpresentations/FilterPapSuctMeasDemonstrtn.pdf

Cementation Skanska. (2009). *Cementation Skanska; Jet Grouting: Technical Data*. Retrieved August 12, 2010, from Cementation Skanska Web Site:
http://www.skanska.co.uk/upload/Sevices/Piling/Datasheets/Jet%20Grouting%20Datasheet.pdf

Chida. (1982). Development of dry jet mixing methods. *Public Works Research Institute, Ministry of Construction* , Japan, pp. 29-35.

Contractor (2009) - Private communications

D. Muir Wood, (2000). Group effects in stone column foundations - model tests. *Geotechnique , Vol. 50* (No. 6), pp. 689-698.

D. G. Fredlund & Harianto Rahardjo (1993). Measurement of Soil Suction. In Delwyn. G. Fredlund & Harianto Rahardjo, *Soil mechanics for unsaturated soils* (pp. 77-80). John Wiley & Sons, Ltd.

DGI-Menard Inc. (2009). *Services - Columns - Controlled Modulus Columns*. Retrieved October 27, 2009, from DGI-Menard Inc - Ground Improvement Specialists Web Site:
http://www.dgi-menard.com/cmc.html

Domone, J. M. Illston & P. L. J. “Portland cements.” In *Construction materials: their nature and behaviour*, by J. M. Illston & P. L. J. Domone. London: Spoon Press, 2001.

Earth Tech. (2010). *Earth Tech; Services and Solutions; Jet Grouting*. Retrieved April 5, 2010, from Earth Tech Web Site:
http://www.earthtech.net/commercial/services_solutions_jetgrouting.php

Egan, J. (1998). *Rethinking Construction: Report of the Contruction Task Force.* London: HMSO.

Fang, H.-Y. (1990). *Foundation Engineering Handbook; Second Edition.* Massachusetts: Van Nostrand Reinhold.

Gastaldi, C. (2009, August 28). Ettringite and calcium sulfoaluminate cement: investigation of water content by near-infrared spectroscopy. *Jounal of Materials Science, 44*, pp. 5788-5794.

Grutzeck, S. K. (1999). A sorosilicate model for calcium silicate hydrate (C- S- H). *JOURNAL OF MATERIALS SCIENCE LETTERS, 18* (3), pp. 217-220.

Hall, C. (1989). Water sorptivity of mortars and concretes: a review. *Magazine of Concrete Research , 14* (147), pp. 51-61.

Holm, G. (1999). Application of Dry Mix Methods for Deep Soil Stabilization. In G. H. Håkan Bredenberg, *Dry Mix Methods for Deep Soil Stabilization* (pp. 3-16). Rotterdam: A.A.Balkema.

Hryciw, S. L. (2007). Evaluation and Quality Control of Dry-Jet-Mixed Clay Soil-Cement Columns by Standard Penetration Test. *Transportation Research Record: Journal of the Transportation Research Board , 1849* (2003), pp. 47-52.

Huat, A. &. (2005, January 1). Field and Laboratory Suction- Soil Moisture Relationship of Unsaturated Residual Soils. *American Journal of Environmental Sciences* , pp. 34-40.

Huat, S. K. (2009). Assessment and Comparison of Grouting and Injection Methods in Geotechnical Engineering. *European Journal of Scientific Research , 27* (2), pp. 234-247.

Ingles, O. G. (1987). Soil Stabilization. In F. G. Bell, *Ground Engineer's Reference Book* (pp. 1-13). London: Butterworth and Co Ltd.

Jones, R. M. (2010) - Private communications

Larsson. S (2005). State of practice report - Execution, monitoring and quality control. *Proceedings from Dry mix methods for deep soil stabilisation; Volume 2* , pp. 732-786.

Larsson, D. (2005). Uniformity of lime-cement columns for deep mixing: a field study. *Proceedings of the ICE - Ground Improvement, 9* (1), pp. 1-15.

Leong, R. B. (2008, December). Indirect Measurement of Suction. *Geotechnical and Geological Engineering, 26* (6), pp. 633-644.

Liu, J. Z. (2008). Statistical Evaluation For Strength of Pile By Deep Mixing Method. In N.C.–2. Proceedings of the 2nd International Conference GEDMAR08, *Geotechnical Engineering for Disaster Mitigation and Rehabilitation.* Nanjing: Springer Berlin Heidelberg.

Lu, N. (2008). Is Matric Suction a Stress Variable? *JOURNAL OF GEOTECHNICAL AND GEOENVIRONMENTAL ENGINEERING, 134* (7), pp. 899-904.

McCabe. (2009, December). A review of field performance of stone columns in soft soils. *Proceedings of the Institution of Civil Engineers , Geotechnical Engineering Vol. 162* (GE6), pp. 323–334.

McCabe, M. (2007, March 15). *GROUND IMPROVEMENT USING THE VIBRO-STONE COLUMN TECHNIQUE.* Retrieved April 7, 2010, from THE INSTITUTION OF ENGINEERS OF IRELAND: http://www.engineersireland.ie/media/engineersireland/community/whitepapers/Ground%20Improvement%20using%20the%20Vibro%20Stone%20Column%20Technique.pdf

McKelvey, V. S. (2004, March 25). Modelling vibrated stone columns in soft clay. *Proceedings of the Institution of Civil Engineers , Geotechnical Engineering Vol.157* (GE3), pp. 137–149.

McLeod, R. (2005). *Ordinary Portland Cement.* Retrieved August 13, 2010, from Building for a Future: http://www.buildingforafuture.co.uk/autumn05/ordinary_portland_cement.pdf

Menard. (2010). *Techniacl Brochure CMC*. Retrieved July 15, 2010, from Menard: http://www.menard.ae/pdf%20files/technical%20brochure%20CMC.pdf

Miao, W. Z. (2009). Experimental Study on Controlled Modulus Column Methods. *Advances in Ground Improvement: Research to Practice in the United States and China (GSP 188),* (pp. 112-119). China: American Society of Civil Engineers.

Minerals, I. P. (2008, March). *European Performance Minerals: Products: Kaolin*. Retrieved July 07, 2010, from Imerys Performance Minerals Website: http://www.imerys-perfmins.com/uploadfiles/44/DAT002K_Speswhite.pdf

Muir Wood, D. (2009). Strength. In D. Muirwood, *Soil Mechanics: A One-Dimensional Introduction* (pp. 193-195). Cambridge University Press.

Muir Wood, D. (2009). Consolidation. In D. Muirwood, *Soil Mechanics: A One Dimensional Introduction* (pp. 138-149). New York: Cambridge University Press.

Munfakh, G. A. (1997). Ground improvement engineering - the state of the US practice: part 1. Methods. *Ground Improvement , 1*, pp. 207-208.

Neville, A. M. (2005). *Properties of Concrete; Fourth and Final Edition.* Essex: Pearson Education Limited.

Porbaha, S. L. (2006). Design and Monitoring of an Embankment on Controlled Modulus Columns. *Jouranl of the Transport Research Board, No 1975: Soil Mechanics* , pp. 96-103.

Quillin, K. (2007). BELITE-BASED CEMENTS. In c. p. CALCIUM SULFOALUMINATE CEMENTS: Reduced CO2, *Keith Quillin* (pp. 16-19). Watford: IHS BRE Press.

R. Bulut, R. L. (2001). Suction Measurements by Filter Paper. *Expansive Clay Soils and Vegetative Influence on Shallow Foundations* (pp. 243-261). Houston: The Proceedings of the

Geo-Institute Shallow Foundation and Soil Properties Committee Sessions at the ASCE Civil Engineering Conference.

Rahardjo, D. G. (1993). Measurements of Soil Suction. In D. G. Rahardjo, *Soil mechanics for unsaturated soils* (pp. 64-79). John Wiley & Sons.

Stavridakis, E. (2006). A Solution to the Problem of Predicting the Suitability of Silty–clayey Materials for Cement-stabilization. *Geotechnical and Geological Engineering , 24* (2), pp. 379-398.

Taylor, H. F. W. "Structure and properties of Portland cement pastes." Chapter 7 in *Cement Chemistry*, by H. F. W. Taylor, pp. 231-247. London: Thomas Telford Publishing, 2004

TerraSystems Inc. (2010). *Home: FAQ: Stone Columns*. (TerraSystems, Inc.) Retrieved July 2010, 17, from TerraSystems Incorporated: http://www.terrasystemsonline.com/faq/stone-columns.html

Topolnicki, M. (2004). In situ soil mixing. In M. P. Kirsch, *Ground Improvement; 2nd edition* (pp. 346-353). Oxon: Spon Press.

Verghese, S. (1991). Concrete Permeability - A Review of Process and Mechanisms. In K. Weshe, *Fly Ash in Concrete* (2nd Edition ed, pp. 97-100). Taylor and Francis.

Wehr, W. S. (2004). Deep Vibro Techniques. In M. M. Kirsch, *Ground Improvement* (2nd Edition, pp. 57-80). London: Spon Press: Taylor and Francis Group.

West, D. R. (1997). Lime-stabilised soil columns. In D. R. G. West, *Review of lime piles and lime-stabilised soil columns* (pp. 8-10). Berkshire: Thomas Telford.

Whitlow, R. (1995). Origins and Composition of Soils. In R. Whitlow, *Basic Soil Mechanics: Third Edition* (pp. 5-12). London: Longman Group Limited.

Woodward, J. (2005). Ground Improvement: Vibro-replacement - Stone Columns. In J. Woodward, *An introduction to geotechnical processes* (pp. 68-69). Oxon: Spon Press.